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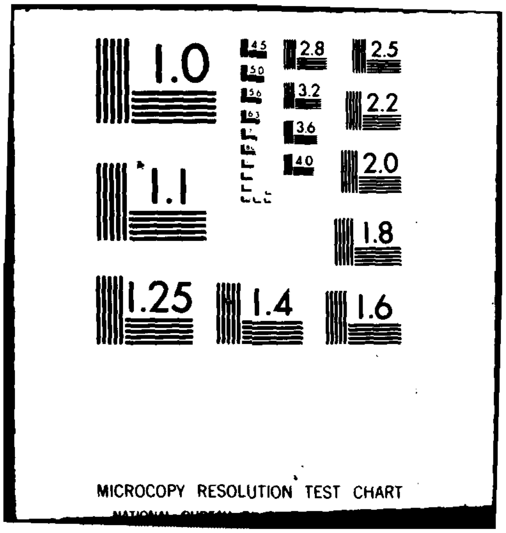
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Hydrologic Analysis of Ungaged Watersheds Using HEC-1 April 1982

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This report discusses general approaches to frequency analyses for ungaged watersheds, effects of the extent of data availability on choice of approach, and regionalization of hydrologic parameters. Hypothetical rainfall data are often the only rainfall data available for a study, so methods for use of this type of data are presented. The use of HEC-1 for watershed modeling is described, and techniques are given in detail for the estimation and calibration of HEC-1 model parameters.

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HYDROLOGIC ANALYSIS OF UNGAGED WATERSHEDS
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PREFACE

This report presents guidelines and procedures for hydrologic investigations of "ungaged" watersheds, that is, watersheds for which available streamflow data are insufficient for making dependable discharge-frequency estimates by statistical procedures. Data availability may vary from the extreme of none at all to situations where some discharge-frequency data can be used with a hydrologic model to extend the discharge-frequency curve to estimate less-frequent events. The hydrologic model can also be used to develop discharge-frequency relationships at ungaged locations in the basin.

The procedures presented center on computer program HEC-1, a general-purpose watershed-modeling program package developed at the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. The analysis techniques, developed at HEC, have been used in a number of studies of ungaged watersheds. Examples from three studies are used herein to illustrate use of the procedures.

This report discusses general approaches to frequency analyses for ungaged watersheds, effects of the extent of data availability on choice of approach, and regionalization of hydrologic parameters. Hypothetical rainfall data are often the only rainfall data available for a study, so methods for use of this type of data are presented. The use of HEC-1 for watershed modeling is described, and techniques are given in detail for the estimation and calibration of HEC-1 model parameters.

ACKNOWLEDGMENTS

This report is the result of efforts of a number of people at the Hydrologic Engineering Center (HEC). Johannes J. DeVries prepared the report. The basic outline was developed by David Ford, with contributions from a number of the Center's staff. The chapter on Calibration of Model Parameters (Chapter 5) was written by Dr. Ford. Gary Dyhouse, Chief of the Hydrologic Engineering Section of the St. Louis District, Corps of Engineers, prepared the material in the Appendix while on assignment at HEC in 1979.

The major portion of the material in this report is based on notes, procedures, and special-assistance reports prepared by the Special Assistance Branch of the Hydrologic Engineering Center, much of which was the work of Dale Burnett, past Branch Chief. The review comments of Bill Eichert, Dale Burnett, Arlen Feldman, John Peters, and David Ford were very helpful. William Johnson offered a number of valuable suggestions on both content and organization. Also greatly appreciated is the help of the Training and Methods Branch clerical staff, Eileen Tomita, Diane Harris, and Penni Baker. Ms. Baker provided a great deal of help with the report figures, besides typing drafts. The final typing was done by the Word Processing Center of the Sacramento District, Corps of Engineers.

Director of the Hydrologic Engineering Center during the study period was Bill S. Eichert. Funding for the report was provided by the Office of the Chief of Engineers, U.S. Army Corps of Engineers under the Analytical Techniques Research Subarea of the Corps of Engineers Research and Development Appropriations.

CHAPTER I

INTRODUCTION

1.1 Scope of Report

This report presents guidelines and methods for determining discharge-frequency relationships for ungaged watersheds. It considers the following topics:

- General approaches to frequency analysis
- Effect of data availability on choice of approach
- Use of hypothetical precipitation data for ungaged watersheds
- Basin modeling with HEC-1 and calibration of model parameters
- Regionalization of model parameters
- Case studies to illustrate use of the methods

Emphasis is placed on the use of computer program HEC-1 for the hydrologic calculations. Computer program HEC-1 is a comprehensive, single-event precipitation-runoff model (Hydrologic Engineering Center, 1981, 1980). The program was developed by the Hydrologic Engineering Center (HEC) of the U. S. Army Corps of Engineers in Davis, California. HEC-1 is maintained and supported by the Hydrologic Engineering Center and is widely available throughout the United States at commercial computer facilities, as well as on Corps of Engineer and other federal computer systems.

In HEC-1, the transformation from rainfall excess to streamflow is accomplished either through unit hydrograph or kinematic wave routing procedures. A variety of techniques can be used for calculation of watershed interception and infiltration (refered to as loss rates). The loss rate determination is an important part of the hydrologic analysis; the accuracy of the loss calculations greatly affects the accuracy of the final results of the modeling effort.

The hydrograph parameters and loss rates must usually be determined by calibration, from rainfall and flow data. When no data are available for a

watershed, they must be estimated by using data from similar watersheds. If the watersheds are hydrologically similar, direct transfer of the data may be done with minor adjustments. More generally, regionalization techniques are used to develop runoff transformation and loss rate parameters.

Information on some common methods for the development of hypothetical storms is presented in detail (See Chapter 4 and the Appendix), with examples of the development of storm rainfall distributions. In many studies, hypothetical rainfall data are the only rainfall data available for the study. The approaches presented for storm development use the rainfall intensity-duration-frequency criteria developed by the National Weather Service, as presented in the NOAA Atlases (NOAA, 1973) and in Technical Paper 40 (U.S. National Weather Service, 1961). Corps of Engineers procedures for configuring the hypothetical storms are used. It should be noted that not all areas of the country can use the methods described here, however.

Because the methods presented in this report rely on the features available in computer program HEC-1, detailed techniques are given for the estimation and calibration of HEC-1 model parameters. Included are discussions of the runoff transformation and loss-rate parameters as well as streamflow routing parameters. Typical values of parameters are given for comparison, and the sensitivity of results to parameter estimation errors is discussed.

Three case studies are presented to illustrate the practical application of the material in this report. An example is given for each of three degrees of data availability: (1) for the case where some discharge-frequency data are available; (2) where some rainfall and streamflow data are available, but where streamflow data are insufficient to determine discharge-frequency curves by statistical analysis; and (3) where no streamflow data exist for the watershed.

The primary objective of this report is to describe how HEC-1 can be used to develop peak-discharge-frequency estimates for ungaged areas. The hydrologist can use the material presented here to help choose the

appropriate level and detail of a study, basing this decision on the availability of the data, time and funds available, and the accuracy requirements of the study results. It should be kept in mind that any data for the basin which are available should be used in the calibration of the hydrologic model of the basin and in the development of discharge-frequency curves at gaged locations. By using all the resources available, including data from regional sources, in conjunction with the analytical techniques described here, the hydrologist can usually make reasonable determinations of discharge-frequency relations for ungaged basins.

1.2 Need for Guidelines

A lack of meteorological and hydrological data is one of the greatest obstacles to accurate discharge-frequency analysis (Burnham, 1980). The hydrologist is usually faced with the problem of having few rainfall measuring sites and even fewer streamflow measuring stations in the basin under study. Techniques are therefore required for predicting or forecasting the peak flow rate and/or volume of runoff for events or situations of interest that do not rely on historical records as a check.

Accurate prediction of streamflows is essential in the planning and design of all types of water resource systems. Of major concern is the prediction of the magnitude of flood peaks and their frequency of occurrence. Information on peak flow is needed for the sizing of channels, bridge waterways, storm drainage systems, reservoir spillways, and other hydraulic structures. The frequency associated with a given flood flow permits the selection of the appropriate level of probability for design. General areas for which this information is used include: Design of urban drainage systems, flood plain mapping, and design of structural and nonstructural measures for reduction of flood damages.

Hydrologic studies to provide data for flood insurance studies require flow and frequency estimates for a wide range of recurrence intervals. In these studies, the 10-, 50-, 100-, 500-year (the 10-, 2-, 1-, 0.2- percent events), and the Standard Project Flood are frequently determined. For reservoir and spillway design the largest expected floods are of greatest concern, and the 100-year, 500-year, Standard Project Flood, plus the Probable Maximum Flood magnitudes are required.

CHAPTER 2

GENERAL APPROACHES TO FREQUENCY ANALYSIS FOR UNGAGED BASINS

The method used for hydrologic analysis of an ungaged basin depends upon many factors. These include:

(A) Type of information required:

(1) Peak flow. Only the maximum discharge at some point in the basin is needed in many cases.
(2) Volume of flow. The total storm runoff is required in some circumstances.
(3) Rate of runoff over an extended period of time. A complete hydrograph for a given frequency flood must be specified in many cases.

(B) Type of data available:

(1) Precipitation. Data may be available only for adjacent basins.
(2) Basin characteristics. Required for basin modeling or for regionalization analysis.
(3) Observed or computed peaks for major events. Although no gage exists for long term records, some data may be available for isolated events.
(4) High water marks of major historical events. These can be used for calibration of a basin model.

(C) Resources.

(1) Experience of the hydrologist, familiarity with alternative methods, and access to computer facilities.
(2) Study time-frame and manpower available.
(3) Funds available for the study. This may be the most important factor in influencing how the study is done.

The approaches used include regional frequency analysis, continuous precipitation-runoff analysis with historical precipitation data, and single-event precipitation-runoff analysis using synthetic precipitation data. These three general approaches are discussed in this chapter. There are other widely used procedures, such as USGS nomographs, empirical equations, and others. The reader is referred to McCuen, et al. (1977) for a list of references on other methods.

2.1 Generalized Rainfall-Runoff Relations

Hydrograph analysis concepts may be applied to ungaged watersheds through the development and application of generalized functions for estimating the amount of precipitation lost due to interception and infiltration (loss rates), unit hydrographs, and base flow. The unit hydrograph is usually assumed to give a unique relationship between rainfall excess and surface runoff for a basin regardless of storm size, losses, or other factors (Hydrologic Engineering Center, 1973b). Because of its ease of use, the unit hydrograph has received the most attention by hydrologists. However, other methods of hydrograph generation are also becoming widely used (Feldman, 1979), such as the kinematic wave approach to basin modeling, also a feature of HEC-1. Both the unit hydrograph and the kinematic wave analyses will be discussed in this report.

The determination of loss rates is a major problem because loss rates are extremely variable and are dependent on both precipitation patterns and basin characteristics. Loss rates should correspond to those considered reasonably likely to occur during the given magnitude storm and should be estimated on the basis of observed loss rates for floods that have occurred in the study basin or in similar areas (USCE, 1965; HEC, 1975b). A "criteria" approach where loss rates are chosen to give conservatively low values has also been used. The criteria approach is most appropriate for computing large floods (from synthetic rainfall), such as floods for spillway design, since losses tend to be small relative to rainfall for large and rare events. This is especially true when the ground is frozen at the beginning of the flood. This approach is less appropriate when it is used to compute runoff for a range of flood events for system performance evaluations, due in

part to the fact that different combinations of rainfall intensities and loss rates can yield similar runoff quantities.

Relating loss rates and analytical loss-rate functions to soil type, land use and cover, antecedent precipitation, and rainfall intensity can be done; however, good results have not always been obtained. A commonly used procedure is that of the U.S. Soil Conservation Service (SCS) which permits generalizing loss-rate functions through the SCS curve number technique. The curve number reflects land use, cover, and soil types with allowances for antecedent moisture conditions (U.S. Department of Agriculture, SCS, 1957).

Unit hydrographs for ungaged basins (also known as synthetic unit hydrographs) are usually developed in two steps. First, an equation or procedure is devised that will allow a unit hydrograph to be computed. Second, the procedure (or equation) must be related to definable basin characteristics. The simplest and most direct method is to transfer a unit hydrograph from an adjacent gaged basin of similar hydrological and meteorological characteristics with simple adjustments. This technique is fairly common; however, it is usually difficult to locate "similar" basins. More complicated and more general procedures include deriving parameters that describe the unit hydrograph, and then transferring these parameters with some adjustments. The two most common synthetic unit hydrograph methods used in the Corps of Engineers are the procedures of Clark (1945) and Snyder (1938).

An alternative to the unit hydrograph approach is the simulation of the most significant watershed processes, such as interception and infiltration, overland flow, and channel flow, using small elements of the watershed to trace the movement of water through the basin. The various watershed elements are linked together to produce a model of a complete watershed. The Stanford Watershed Model and the MITCAT Model (MIT Catchment Model) are two well-known examples of this type of model. Models of this type have been designated as "distributed-parameter" models in contrast to "lumped-parameter" models such as the unit hydrograph.

The distributed-parameter approach is very attractive for modeling ungaged basins, because it is based in principle on the use of physically-based parameters for describing the response of the basin to rainfall. The required data include lengths of overland flow paths and overland flow resistance coefficients, channel geometry and roughness, channel lengths, and loss-rate parameters. The channel parameters can be obtained quite readily; however, it is somewhat more difficult to estimate the overland flow parameters. In general, the model should be calibrated using measured flows to ensure its proper formulation.

To permit basin modeling by a distributed-parameter approach, computer program HEC-1 contains kinematic-wave procedures for computing subbasin outflow hydrographs (Hydrologic Engineering Center, 1981). These kinematic wave options provide an alternative to the unit hydrograph method for determining direct runoff.

A description of the kinematic-wave techniques available in HEC-1 is given in Chapter 7. Procedures for computing flow hydrographs are presented. The development of frequency estimates for this approach is in general the same as when the unit hydrograph approach is used, once the hydrographs have been determined for various rainfall frequencies.

2.2 Regional Analysis of Watershed Characteristics

It is sometimes possible to transfer information from watersheds for which data are available to nearby ungaged watersheds within a given region, if they have very similar hydrological characteristics. If unit hydrograph and loss rate parameters can be related to basin characteristics, then a rainfall-runoff model such as HEC-1 can be used to estimate discharge information.

If a regional approach is used, care should be taken to select basins that are indeed similar in hydrological characteristics. Thus, the basins should have similar geological characteristics, topography, land uses, vegetative cover, and agricultural practices. The basins should be of the same general size, and the rainfall distribution and magnitude, as well as the factors affecting loss rates, should be generally the same.

Basic Steps in a Regional Study Using the Unit Hydrograph Procedures of HEC-1:

1. Collect precipitation and runoff information for a range of major flood events in the region. Identify the watershed and subbasin boundaries on a topographic map. Index points where discharge-frequency estimates are desired should be located. The subbasins are chosen to include these index points, desired hydrological components, and stream gage locations. Basins adjacent are also used if they are hydrologically and meteorologically similar.

2. Perform an HEC-1 rainfall-runoff analysis to optimize the unit hydrograph and loss-rate parameters for the gaged drainage areas.

3. Correlate the unit hydrograph and loss-rate parameters with basin characteristics and develop generalized relationships for these parameters.

4. Divide region into subbasins corresponding to their hydrologic characteristics and to the index points at which discharge-frequency estimates are needed, and also to capture the spatial variations in precipitation.

5. The generalized relationships developed in Step 3 are used to compute parameters for the ungaged subbasins using measurable subbasin characteristics.

6. A watershed simulation using HEC-1 is next performed for several historical storm events. If the reconstituted runoff peaks and volumes at downstream gages are significantly different from observed hydrographs, the unit hydrograph, loss-rate, and/or routing parameters must be adjusted and the analysis repeated.

2.3 Continuous Simulation with Historical Precipitation Data

An alternative method to the single event modeling approach is continuous simulation with long term precipitation data after calibrating the model with several years of continuous data. This approach could be used

with any continuous simulation model, such as the HSP model (Hydrocomp, Inc., 1976; Crawford, 1971).

In many cases, the length of record for streamgage data is only a few years. Rainfall data are often available for much longer periods for the same watershed, and hourly data in the U.S. are frequently available for a period of 35 to 70 years. By simulating a streamflow record with these data, sufficient data for a discharge-frequency relationship can be obtained. This relationship is based on the response of the basin during the calibration period, and thus no correction for changes in basin characteristics over the period of record need to be made unless it is known that the discharge-frequency relationship has changed due to urbanization or other effects, such as channelization or water storage projects (Cermak, 1979).

This method has also been used on completely ungaged basins when data are available from adjacent basins with known characteristics to establish the modeling parameters. This type of model requires a similar subdivision of the basin into small subbasins to properly account for the various soil and land use characteristics within the basin. The chief disadvantage of models of this type is the high cost associated with their use. Several factors contribute to these costs: a great amount of data is needed; the personnel time required for setting up the model is significantly greater than for other models; and more computer time is also needed.

2.4 Single Event Simulation with Hypothetical Precipitation Data

In many cases, neither streamflow nor precipitation data records are long enough to determine flood-flow return periods that are less frequent than a few years. If it is assumed that the frequency of a given flood is the same as the frequency of the storm producing the flood then hypothetical storms can be developed on the basis of meteorological analyses to give rainfall data for a particular frequency of occurance.

Analysis of total storm rainfall data on a regional basis permits the development of generalized rainfall-frequency curves for relatively large areas. These are usually based on point rainfall-frequency analyses which are adjusted to account for the areal extent of the storms. These generalized storms are usually refered to as "design storms."

The total storm rainfall can then be distributed in time and used as precipitation input to a hydrologic model (such as HEC-1) to determine the runoff associated with a storm of a given frequency. If the discharge for a particular frequency storm is known, the model loss rates can be adjusted so that the computed discharge matches the observed. A similar adjustment procedure can be used for storms of other frequencies. A number of storms, each of a different frequency, are used to produce a discharge-frequency curve. This is done by either assuming that the rainfall and runoff frequency relationships are the same or by using an adjustment procedure to relate rainfall and discharge frequency. If no adjustment is used, it is assumed, for example, that the 1 percent frequency storm produces the 1 percent frequency flood flow.

The major assumption of this method is that rainfall of a given frequency will produce a flood of the same frequency. It should be recognized, however, that different combinations of rainfall intensities and loss rates can give rise to the same peak discharge. However, when no rainfall or flow data are available, this may be the most acceptable approach. Because of the wide use of this method, the development of hypothetical storms will be described in some detail in Chapter 4.

2.5 Selection of A Particular Method

The particular approach used for the hydrological analysis of a watershed depends on a number of factors (Burnham, 1980). The type of study being performed and the information that is needed from it are important, as well as the amount of time and the personnel available. The characteristics and location of the watershed will influence the method used. Often the experience of the hydrologist and his familiarity with a particular technique will influence him to select a given method over others, even though a less familiar method may give superior results. However, if one assumes comparable expertise, the most important factors in determining the method used for the analysis of an ungaged watershed are the type and amount of data available and the funds available. The following chapter discusses in detail the effect of data availability and will serve as a guide for establishing the methodology to be used in a given study.

CHAPTER 3

EFFECT OF DATA AVAILABILITY ON CHOICE OF APPROACH

It is the rule, rather than the exception, that the data available for a particular hydrologic study are not sufficient to give a complete solution to the problem. The available methods of hydrologic analysis are frequently more precise than the data to support them, even though many hydrologic processes are not fully understood and completely defined. As a consequence, the hydrologist is often constrained to adopt procedures that are not as complete or as rigorous as desired, but that must be used as a result of the type of data that are available. This chapter discusses the degrees of data availability, the general approach to basin analysis using HEC-1, and the influence of data availability on the study procedure.

3.1 Degrees of Data Availability

Three levels of data availability are considered here: (1) Sufficient data to calculate discharge-frequency curves at some locations, but not at others; (2) Sufficient rainfall and streamflow data to calibrate precipitation-runoff parameters at some locations, but insufficient streamflow data to derive discharge- frequency curves by statistical methods; and (3) No streamflow data available within the basin.

3.2 General HEC-1 Study Approach with Limited Data

HEC-1 is a generalized precipitation runoff model for single event simulation (Hydrologic Engineering Center, 1981). Either a unit hydrograph or kinematic wave approach is used to transform rainfall values to runoff. The unit hydrograph is most commonly used, and it is assumed that a single unit hydrograph is appropriate for all magnitudes of rainfall excess. Clark, Snyder, or the Soil Conservation Service unit hydrograph methods can be used. Snowmelt can be included, and several loss rate functions are available. A large basin can be subdivided into subbasins, and stream routing from subbasin to subbasin can be performed with any of several hydrologic routing methods.

The basic sequence followed in the situation where no streamflow is available for the basin is shown in Figure 3.1. The basin average rainfall associated with a particular frequency of occurrence is determined, either from historical records or using methods for developing hypothetical-frequency events (see Chapter 4). Basin average loss rates are set. These may be calculated from regional values; they may be estimated through direct transfer from historical events; or other methods may be used.

The computed time distribution of basin-average rainfall excess (i.e., the rainfall minus losses) is then used with a unit hydrograph and the general basin routing model to compute the runoff hydrograph at the basin outlet. The Clark, Snyder, or SCS unit hydrographs may be used with the hydrograph parameters determined by calibration using measured events (see Chapter 5) or by using parameters determined with regionalization techniques (see Chapter 6). Alternatively, the kinematic wave options of HEC-1 can be used to develop a basin model.

This process is repeated for several storms of different frequencies. The computed flow for each event can be associated with the frequency of occurrence of the rainfall, and a discharge-frequency relationship can be established. This discharge-frequency relationship is adjusted to match events of known frequency by adjusting the loss rates.

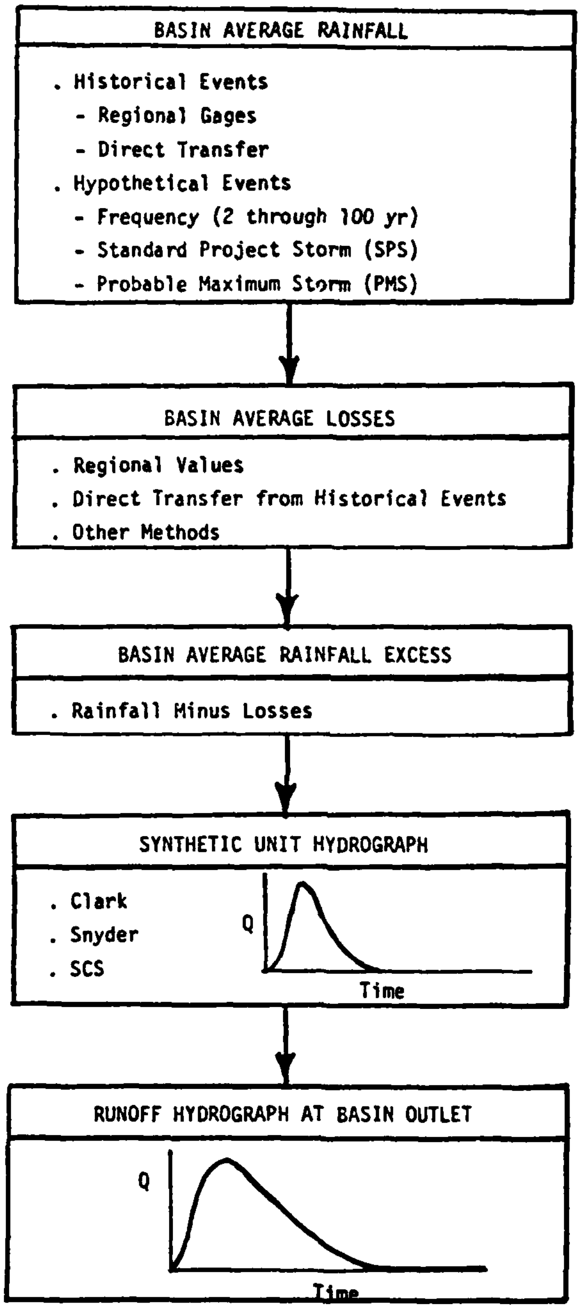
3.3 Influence of Data Availability on Study Procedure

The type and amount of data that are available for the study strongly influence how the study is conducted. This is illustrated here for the case where little or no streamflow data are available. Three possible levels of study procedure are discussed, starting with the most extensive investigation effort.

Procedure A - Complete Study

(1) Obtain Basin Parameters

Determine locations in a hydrologic region similar to the study basin where discharge has been measured during several (usually 5 to 10) major storm events. Collect streamflow and rainfall data for these events at as



3.1. Development of a Runoff Hydrograph for a Simple Ungaged Basin Using HEC-1.

many locations as possible. Determine unit hydrograph or kinematic wave parameters, loss-rate parameters, beginning flow rates, and recession characteristics which do the "best" job of reproducing the observed hydrographs. Adopt representative values of these parameters. Optimization features of the HEC-1 model are normally used for this step.

(2) Regionalize Basin Parameters

Perform regression studies on unit hydrograph parameters and basin physical characteristics using the procedures given in Chapter 6.

(3) Develop Streamflow Frequency Curves for Gaged Sites

If any of the stream gages studied are near the basin of interest, perform discharge-frequency analyses on the annual series of observed maximum discharges.

(4) Model the Study Area

a. Subdivide the study basin into as many subbasins as are necessary to establish index points for each stream reach where discharge-frequency estimates are required. Determine the drainage area and other physical characteristics of each subbasin and adopt unit hydrograph or kinematic wave parameters for each, based on the regional studies of Step (2).

b. Determine channel length between index and/or combining points and estimate channel routing characteristics either by detailed field surveys, stream cross-section and multiple-discharge water surface profile studies, or any acceptable approximate means commensurate with the scope of the study. Adopt channel routing parameters.

c. Assemble subarea data decks for the HEC-1 runoff model for several historical storms and adjust loss parameters and routing parameters to best reproduce the observed hydrographs at gaged points, if gaged points are present.

(5) Develop Hypothetical Events

Develop rainfall frequency curves using the National Weather Service procedures described in Chapter 4, for various rainfall durations (say

durations of 1, 3, 6, 12, and 24 hours). For a selected storm frequency, determine rainfall depths for each duration at desired time intervals, compute incremental depths, and arrange in appropriate order around the maximum value. This is repeated for each of several storm frequencies such as the 50-percent, 10-percent, 2-percent, 1-percent events (i.e., the 2-year, 10-year, 50-year, 100-year recurrance intervals). These become the hypothetical storm inputs. Point rainfall values should be adjusted to the appropriate areal size using the guidelines given in Chapter 4.

(6) Assign Frequency to Hypothetical Events at Gaged Sites (If Available)

If there is a gage at some location either in the basin or on a nearby watershed the data from this gage can be used to develop a better estimate of the discharge-frequency relationships at the locations which have no gages. If there is no suitable gage, this step is skipped.

Use the hypothetical storm data of Step (5) as input to the calibrated model of Step (4) and generate peak discharges at whatever gaged locations may be available for selected rainfall frequencies. Compare the results with discharge-frequency curves at the gaged locations. Adopt indicated frequencies for hypothetical events or adjust loss rates in a logical manner until reasonable agreement is obtained between the two frequency curves.

(7) Determine Point Runoff and Frequency for Desired Locations

Use HEC-1 to generate peak discharges for each subbasin and combining point throughout the basin. If they are available, the loss rates established in Step (6) are used with the hypothetical precipitation data. Separate HEC-1 runs are required for each frequency and rainfall distribution pattern, but the jobs can be stacked so that they are all processed at one time. Plot the peak discharge values vs exceedence frequency on a log-normal probability paper and interpolate to get other discharge frequencies, if they are desired. Compare results with those available from other methods and adopt final values.

Procedure B - Regional Unit Hydrograph Parameters Available

If regression equations for regional unit hydrograph parameters are already available, Procedure A can be modified by omitting Steps (1) and (2).

When there are no observed data within the basin under study, streamflow-frequency curves should be developed for near-by basins by treating the area above each gage as a single subbasin (Step (3)). Step (4c) will also be omitted in this procedure.

Procedure C - Data Available from Previous Studies

If a discharge-frequency curve is desired at only one location and no observed data are available, but regional unit hydrograph and loss rate parameters are available from previous studies, only Steps (5) and (6) are needed. In this situation the basin would not be subdivided. Appropriate loss rates would be subject to considerable judgment, however, and could vary widely, depending on the criteria used by the individual selecting the value.

CHAPTER 4

HYPOTHETICAL PRECIPITATION DATA

4.1 Introduction

Hypothetical rainfall data are used when rain gage data are not available or when records are too short to develop rainfall-frequency relationships. Hypothetical storms are also used as a basis of design for projects which pose a threat to property or would result in loss of life. General use of these data is discussed in this chapter. Detailed guidelines for hypothetical storm development are given in the Appendix.

Sources of Storm Data. The primary sources of hypothetical storm information for the United States are various technical publications (TP) of the National Weather Service (NWS) and the hydrometeorological reports (HR) of the National Oceanic and Atmospheric Administration (NOAA) (NOAA, 1973; NWS, 1961; for example). The development of hypothetical-frequency storms from data given in these publications is based on generalized rainfall maps and regression equations. Other methods, such as statistical analysis of nearby long-record rain gages to derive the hypothetical storms of particular frequencies, are used extensively in some parts of the United States but are not discussed here.

The United States is divided into two major geographical areas based on type of precipitation. The 35 states east of the Rocky Mountain area are essentially free of significant orographic effects and thus are included in one region. The 13 mountain states (those containing the Rocky Mountains and states to the west) are covered on a state-by-state basis as well as in site-specific publications. Similar data are available for Alaska and Hawaii. The NWS procedures are based on statistical evaluations of long-term rainfall-gage records in a region. These evaluations include estimates of the frequency of accumulated rainfall-depth versus storm duration at each rain gage. Rainfall maps made from these depth-duration values were used to define lines on the maps of equal total rainfall within a specific period. These permit consistant rainfall-depth relationships through a region for

specific storm durations to be defined. These lines of equal rainfall depth are termed "isopluvial" lines; a typical isopluvial map is shown in Figure 4.1. Derivation of the rainfall-frequency depth-duration relationships is detailed in each of the publications.

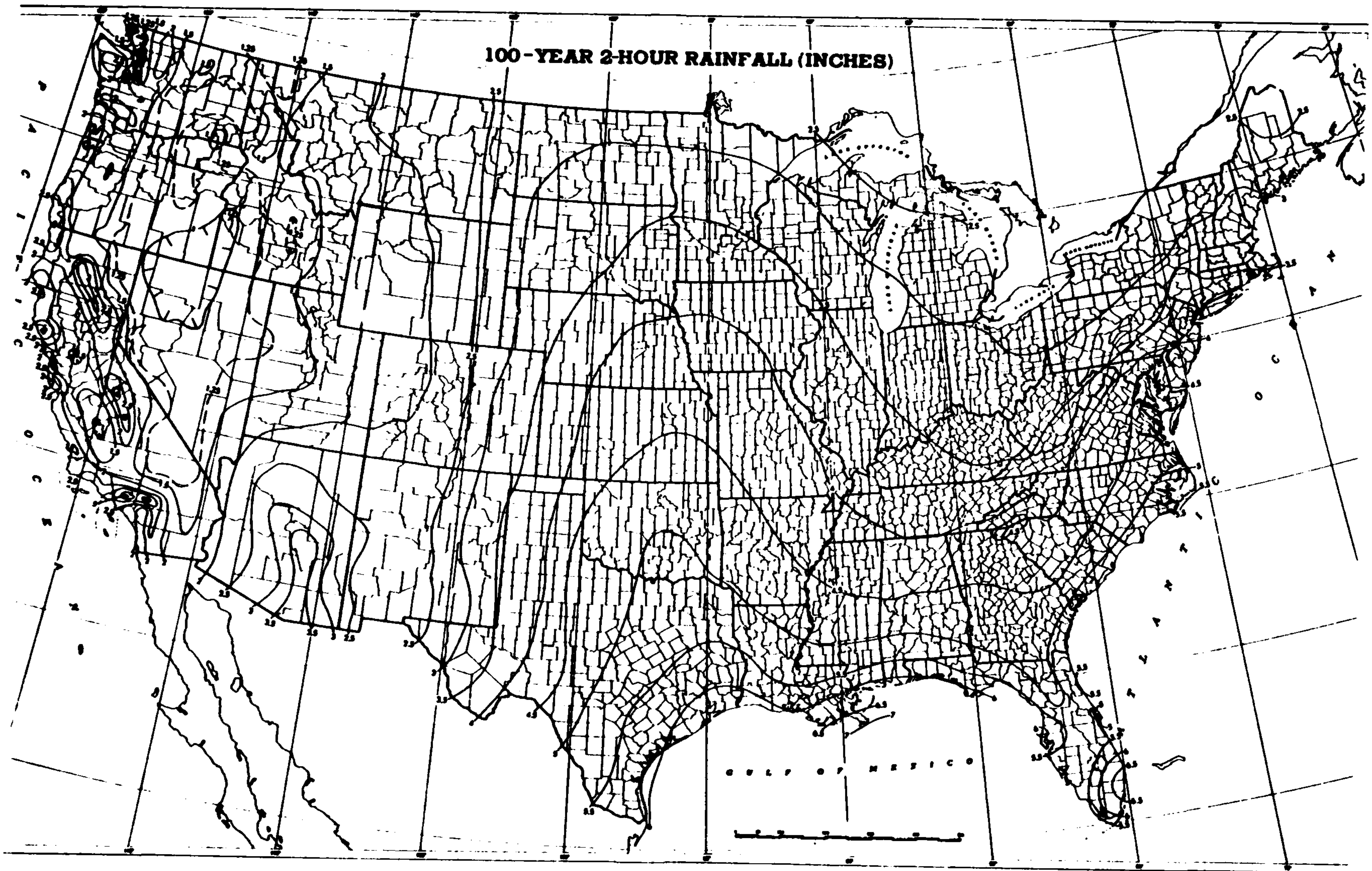
The precipitation-frequency maps given in the NWS and NOAA publications show point-precipitation values that can be assumed to apply to areas up to 10 sq mi. For larger areas, the average precipitation over the area is less than the maximum value at a point, and thus adjustments are required. In general, depth-area-duration relations vary with storm type and intensity and can also vary with the region.

A hypothetical storm developed from NWS data is sometimes called a "balanced" storm, because a consistent depth-frequency relation is used for each duration interval of the total storm. For example, for the 100-year return period, 48-hour duration storm, rainfall depths for a given period (say the 30-minute, 1-hour, 6-hour, or 24-hour periods) would each be equal to the 100-year rainfall depth for that duration interval. This consistent frequency-depth-duration relationship throughout a storm does not occur in nature, of course, because of the random nature of rainfall events. Use of a balanced storm, however, permits the construction and arrangement of a storm event such that an average precipitation intensity of a specified frequency is provided for all durations -- including one that matches the time-response characteristics of the particular watershed being analyzed.

4.2 Types of Hypothetical Storms

The storms determined from National Weather Service criteria as described above give hypothetical storm data for specific frequencies. For example, a storm with an X-year return period has a 100/X percent probability of being equaled or exceeded in any given year. Thus, a 50-year storm has a 2 percent chance of being equaled or exceeded in any given year and would be equaled or exceeded, on the average, once every 50 years.

The other types of storms that are frequently used are the Standard Project Storm (SPS) and the Probable Maximum Storm (PMS). The SPS represents the most severe meteorologic conditions considered reasonably characteristic



4.1. Typical Isopluvial Map

Source: National Weather Service

of the geographic region involved, excluding extremely rare events. The flood resulting from the SPS gives a "standard" against which the performance of a system can be compared with similar systems in other locations.

The Standard Project Storm is used widely within the Corps of Engineers (USCE, 1965). Development of the SPS is described in detail in the Appendix, and example calculations are given. However, because development of the SPS for the Western states requires site-specific criteria, procedures for the Western states are not covered in this report.

The Probable Maximum Storm (PMS) is considered the maximum storm that can be reasonably expected to occur in a given region. Accepted meteorological procedures are used to determine the upper limits of rainfall amounts, which are then assembled into critical chronological sequences. PMS data are usually developed by using depth-area-duration relationships for precipitation from major storms that were measured in the region or could have occured. Precipitation amounts for the storms are adjusted to correspond to maximum moisture conditions and maximum rate of moisture inflow to the storm location. Envelope curves based on the adjusted values for all storms are then used to develop the PMS depth-area-duration curves.

Generalized charts for PMS rainfall are available for the United States from National Weather Service publications. Two publications covering the U.S. in general are NWS (1956) for areas east of the 105th Meridian and NWS (1960) for western areas. Procedures for distributing the probable maximum precipitation estimates in time and space for the United States east of the 105th Meridian are given in NWS (1981). The Appendix gives an extensive list of publications with PMS data for specific areas.

4.3 Applications for Hypothetical Storms

The various types of hypothetical storms are commonly used in planning investigations of flood control components, design analyses, and in flood plain management studies. The types of storms used and their applications depend on the requirements of the study.

Hypothetical storms for various recurrance intervals are used in planning investigations to develop frequency curves for existing and modified conditions as input into economic evaluations. In design analyses a design storm is used to generate the runoff that is selected as the standard against which the performance of a facility can be evaluated. The design flood is simply the runoff from the design storm. Hypothetical frequency storms are used to design channels, storm sewers, agricultural levees, detention areas, and other features.

Hypothetical storms are frequently used in flood plain management studies to develop flood hazard information. Usually, these hypothetical frequency storms are used to generate the flood events of various return periods, such as the 100-yr event for flood insurance studies. The Standard Project Storm is often also included for flood hazard information in flood plain management studies.

Standard Project Storms are used in planning investigations as an upper boundary in determining flood control storage requirements. For project design work the Standard Project Storm is the design storm to be used where some small degree of risk can be accepted but an unusually high degree of protection is required (due to loss of life, high property values, etc.). It is typically used for determination of storage in flood control reservoirs and studies for main river levees in urban areas. The projects for which Standard Project Flood estimates are required are specified in EM-1110-2-1411 (USCE, 1965).

The Probable Maximum Storm is used as a design storm where virtually no risk of failure can be tolerated, such as for most dam spillways. In planning investigations Probable Maximum Storms are generally used in determining spillway size and top of dam elevation.

4.4 Construction of Hypothetical Storms

When constructing a hypothetical storm, it is necessary to establish the appropriate storm duration and the time interval for subdividing the storm rainfall. One can then take rainfall values from the appropriate NWS publication, make adjustments to compensate for size of drainage area, adjust

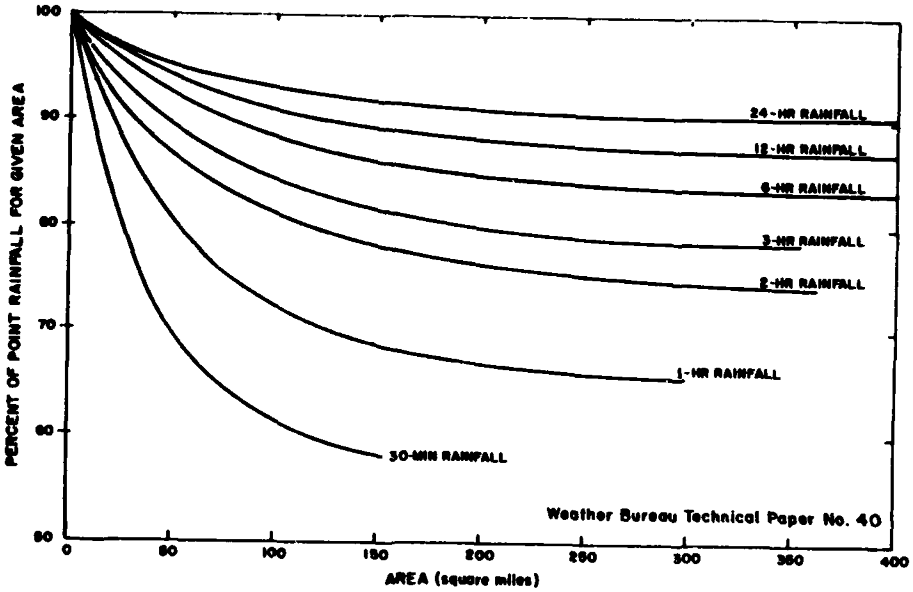
from partial to annual series (if required), and finally increment and arrange the storm rainfall for later use with the hydrologic model of the watershed. Each of these steps is described in detail with examples in the Appendix.

The storm duration and the time increment for calculations are functions of the type and size of the drainage area. The appropriate duration of the storm depends on the basin time of concentration, i.e., the travel time from the upper portions of the basin to the point of interest farthest downstream. The overall duration is dictated by the purpose of the estimate and the importance of total runoff volume in the use of the design flood.

Rainfall frequency curves are derived from TP 40 (NWS, 1961), or other appropriate source, for durations of 2, 3, 6, 12, and 24 hours. The 30-minute and 1-hour maps in TP-40 have been superceded by data given in HYDRO-35 (NWS, 1977) for 15- and 60-minute rainfall depths. The value of the 30-minute rainfall depth is obtained from the 15- and 60-minute values using an equation given in HYDRO-35. The values must be adjusted for annual series, since the TP 40 and HYDRO-35 charts are for partial duration series. For a selected storm frequency, rainfall depths are determined for each duration at desired time intervals and incremental depths computed. If durations greater than 24 hours are needed TP 49 (NWS, 1964) can be used.

The rainfall values from the maps represent point rainfall. However, the average rainfall over a given area will be less than the maximum point value in the area. Figure 4.2, which is taken from TP40, gives the reduction in the point value for various precipitation durations as a function of area size.

The rainfall time-intensity pattern is determined next. Rainfall depth can be plotted as a function of duration on log-log graph paper. If a line is drawn through the points, the plot can then be used to determine accumulated rainfall depths for any desired interval. The accumulated rainfall is divided into increments to determine the amount in each period.

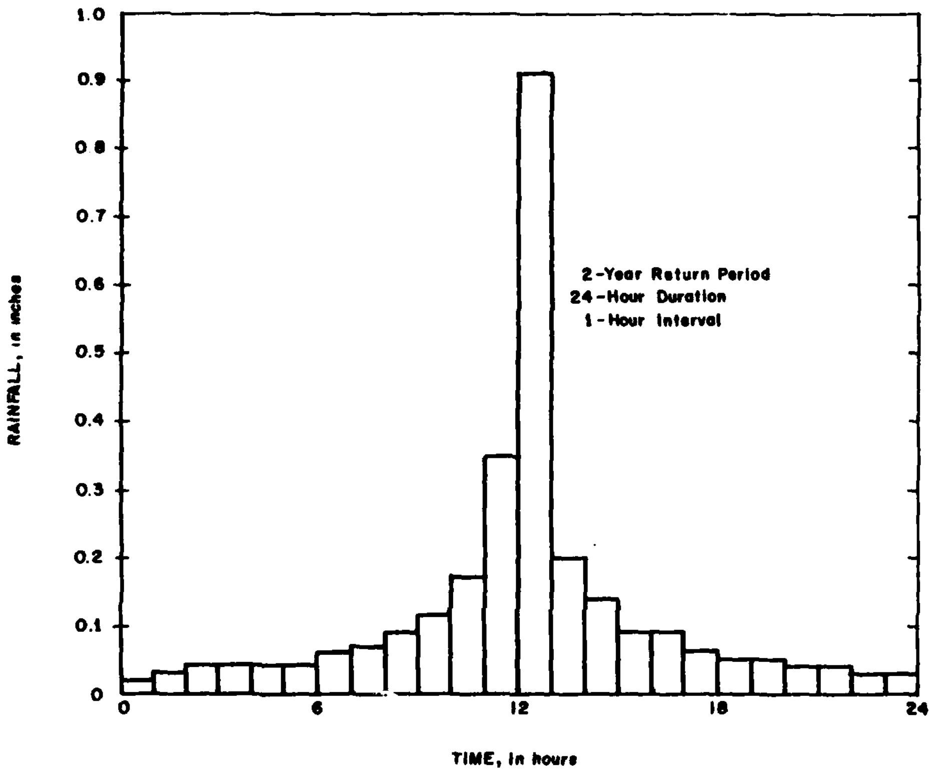


4.2. Adjustment of Rainfall Data for Area

The rainfall values are then ranked by magnitude to produce a rainfall distribution. The following order is most commonly used (USCE, 1965): the greatest rainfall increment is usually placed in the center of the distribution, the next largest is placed in the time increment before it, and the next after. For succeeding values, the next largest is placed before, the next after, and so forth. A storm distribution of this type is shown in Figure 4.3. Other distributions can also be employed. It may be desirable in some cases to analyze actual storm distributions to develop a representative storm pattern for a given region.

If the precipitation time-intensity pattern for a storm with a duration longer than 24 hours is needed, all 24-hour periods outside of the peak 24-hour period can be represented by an average value for each period, because the effects of rainfall variations away from the peak are small (Linsley, Kohler, and Paulhus, 1975).

The Standard Project Storm distribution (USCE, 1965) has been used to develop an alternative arrangement of rainfall in time. However, although the SPS arrangement is sometimes applied to hypothetical storms, it was



4.3. Typical Time Distribution of Storm Rainfall

specifically derived for events that are rarer than the 100-year return-period event. Its application will produce a more severe distribution of rainfall intensity in time than may be reasonable for a hypothetical storm, so excessively high estimates of peak discharge may result when the SPS arrangement is used.

4.5 Consistent Depth-Area Relationships.

It is assumed here that rainfall depths for hypothetical storms are directly related to a particular drainage area size. However, because it is necessary with HEC-1 watershed models to apply rainfall to different subarea sizes to compute runoff from each, different rainfall totals should be applied to each watershed (based on subarea size) to obtain runoff consistent with the particular event under study. This can be done with HEC-1 by employing the Precipitation Depth-Area Relationship. Section 7 of the HEC-1 Users Manual (HEC, 1981) describes this feature in detail. In brief, this option of the program uses a total-storm-depth-versus-area relationship for each runoff, routing, and combining operation of the watershed modeling process to develop at each computation point a hydrograph that is based on a rainfall depth that is consistent with the actual drainage area. A key feature not covered in the manual is that the appropriate rainfall distribution for the subarea under study must be used. The point rainfall total may not differ greatly from that for a 50-square-mile area for the total storm duration, but the distribution will give significantly different incremental values for the short-duration peak periods. The peak periods are greatly reduced as the drainage area increases, as shown in Figure 4.2. One should use several distributions (based on representive drainage areas) for the consistent depth-area option. An appropriate application would be the development of three to five distribution patterns for areas ranging from the smallest subarea to the area of the entire watershed model. The distribution for the area nearest in size to the subarea under analysis should then be specified for each runoff hydrograph calculation.

4.6 Other Considerations

The hypothetical-storm data are used as input to a calibrated hydrologic model of the basin to generate peak discharges at gaged locations for selected rainfall frequencies. The model results are then compared with computed (adopted) discharge-frequency curves at gaged locations. The standard procedure is then to adjust the losses until reasonable agreement is obtained between the two frequency curves. The procedure is based on the assumption that the frequency of a particular storm and loss rate combination

can be directly related to the frequency of the flood produced. This usually gives a reasonable estimate of the relative frequency of various discharges at ungaged locations within a basin.

CHAPTER 5

CALIBRATION OF MODEL PARAMETERS

5.1 Need for Model Calibration

Application of HEC-1 to a study of the precipitation-runoff process in a basin requires calibration of the model; that is, values of the parameters of the numerical model of the physical system must be established. For a strictly physically-based model, these parameters can theoretically be determined by measurement. Practically, however, the parameters are determined most often by using observed precipitation and runoff data to solve the inverse problem. That is, given the system input (precipitation) and the system output (runoff), the inverse problem (and hence the calibration problem) is to define the characteristics of a system that produces the transformation from input to output. With a specific numerical model such as HEC-1, the functions that define the transformation are preselected; calibration requires only selection of the parameters of these transformation functions.

This chapter presents a discussion of the loss rate and unit hydrograph precipitation-to-runoff transformation functions of HEC-1, and it defines the parameters of those functions that are adjusted to model characteristics of a particular basin. HEC-1 has the capability to estimate optimal values of the unit hydrograph and loss-rate parameters automatically; this capability is described here, and a procedure for its use is presented. The program also has the capability to estimate parameters of the routing functions that simulate the motion of water in the streams and channels of a basin, and this is described as well. Determination of the routing parameters is possible only if the shape of the local inflow hydrograph is known or assumed, however. Finally, typical values of the model parameters are suggested.

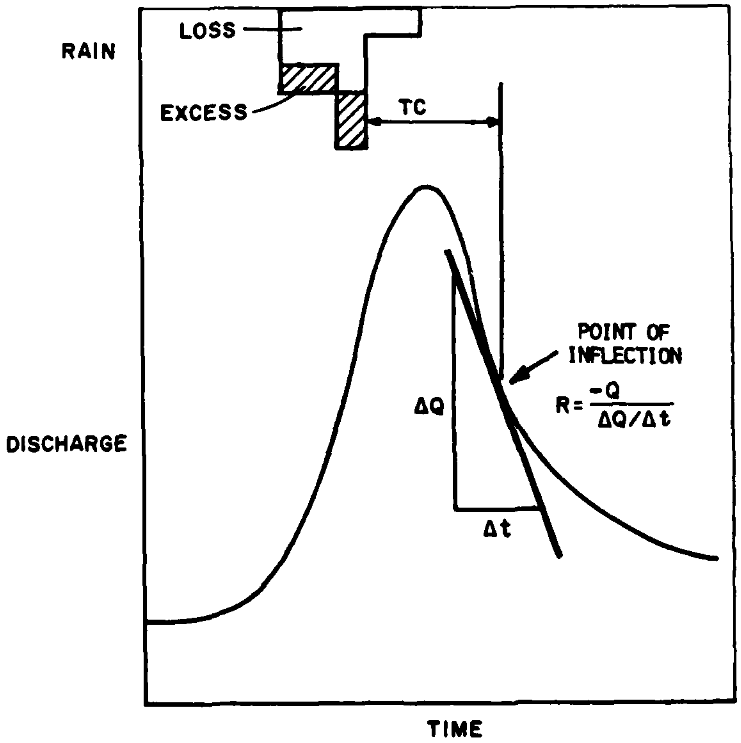
5.2 Unit Hydrograph Specification

HEC-1 is most commonly used to model the precipitation-runoff process by application of the unit hydrograph technique proposed by Clark (1945). This technique defines the ordinates of a unit hydrograph with only two

parameters: t_c, the time of concentration (in the program the designation "TC" is used for t_c), and R, a storage constant. Use of this technique is attractive because it avoids problems inherent in defining, through calibration, individual ordinates of the unit hydrograph. The general shape of the hydrograph is fixed, and problems with negative ordinates and infeasible fluctuations of the unit graph ordinates are eliminated.

In addition to the two parameters, the Clark method uses a time-area relation to define an instantaneous unit hydrograph. The first parameter (t) theoretically is the travel time of a water particle from the most upstream point in the basin to the outflow location. An estimate of this lag time is the time from the end of the runoff producing rainfall to the inflection point on the recession limb of the surface runoff hydrograph (Figure 5.1).

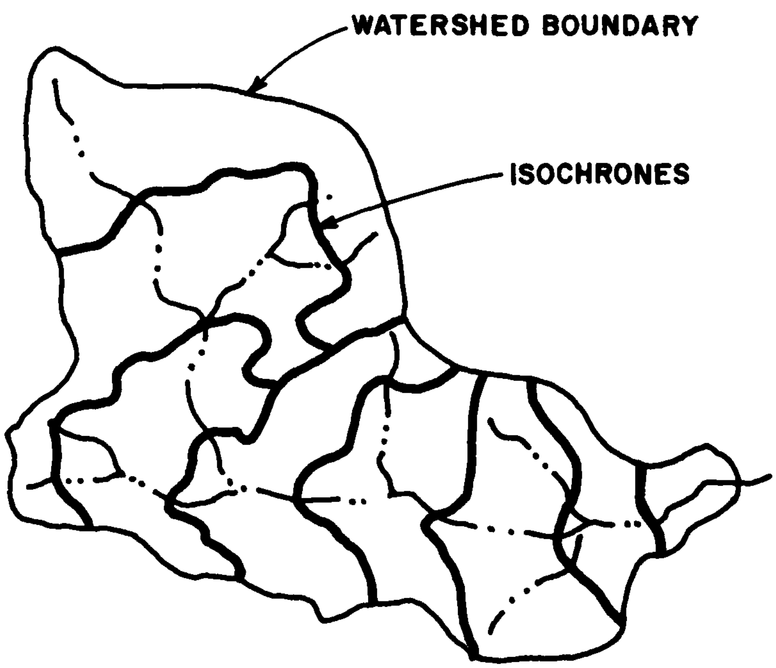
The parameter R also has the dimension of time and is used to account for the effect of basin storage on the hydrograph. This parameter can theoretically be estimated by dividing the flow at the point of inflection of the surface runoff hydrograph by the rate of change of discharge (slope) at this same time (Figure 5.1). Another method for estimating R is



5.1. Illustration of Clark Coefficients

by computing the volume remaining under the recession limb of the surface runoff hydrograph following the point of inflection and dividing this volume by the flow at the point of inflection.

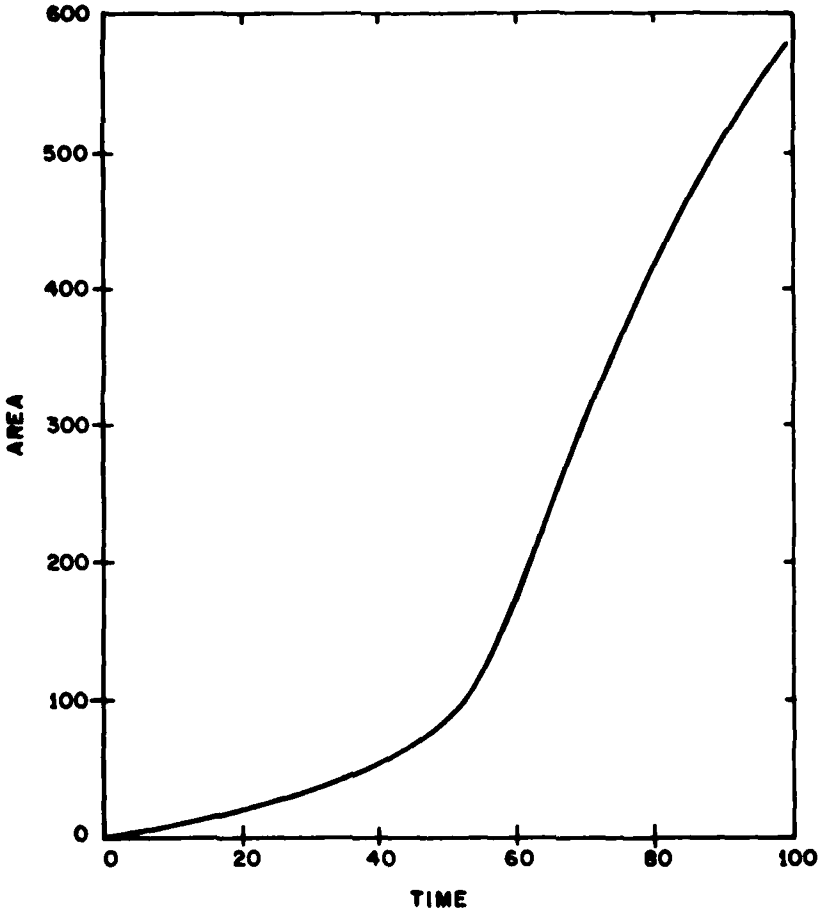
The other item necessary to compute a unit hydrograph with the Clark technique is a time-area relation. If a synthetic time-area curve is not used, the time-area relation is developed by dividing the basin into incremental runoff-producing areas that have equal incremental travel times to the outflow location (Figure 5.2). The distance from the most upstream point in the basin is usually measured along the principal watercourse to the outflow location. Dividing this distance by an assumed value of t_c gives an estimate of the rate of travel. The value of t_c should be for that point in the basin that is furthest from the outlet based on travel time. In some cases this may not be a point that is reached by extending the principal watercourse. Isochrones representing equal travel time to the outflow location are laid out using the rate of travel to establish the location of the lines. The areas between the isochrones are then measured and tabulated in upstream sequence along with the corresponding incremental travel time for



5.2. Division of Basin on Basis of Travel of Travel Time to Outlet

each incremental area. The increment of time used to subdivide the basin need only be small enough to define adequately the areal distribution of runoff, while the time period selected as the computation interval must be equal to or less than the unit duration of precipitation excess. Because the former may be larger than the latter, a plot of percent of time of concentration versus accumulative area is useful in determining time-area relationships for model calibration (Figure 5.3). Such a curve facilitates rapid development of unit hydrographs for various values of the still undetermined parameter, t_c, without requiring development of a new time-area relation. This development is based on an assumption that the percent of time versus accumulated-area relation will remain constant for all values of t_c.

A detailed description of implementation of the Clark unit hydrograph technique in computer program HEC-1 is presented in the HEC-1 Users Manual (HEC, 1981) and in Volume 4 of the Hydrologic Engineering Methods for Water Resources Development Series (HEC, 1973b).

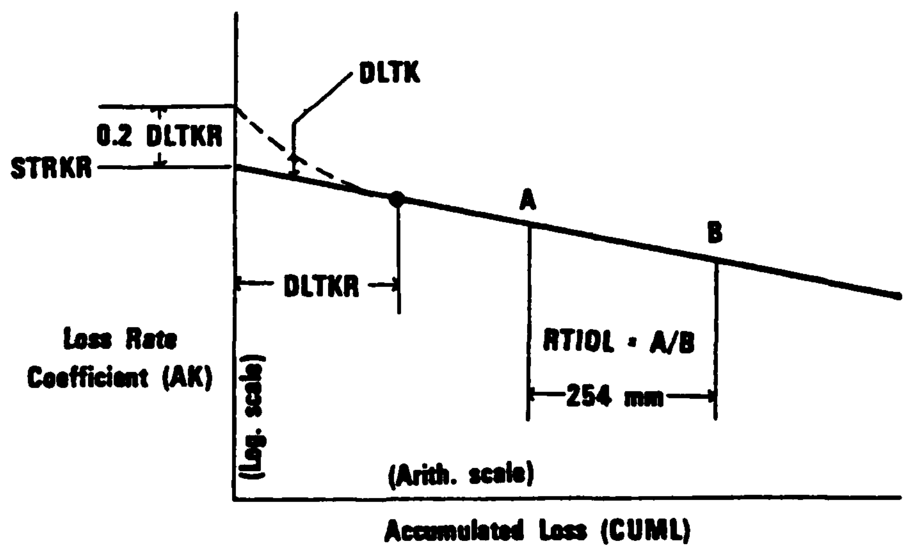


5.3. Typical Watershed Time-Area Relationship

The parameters t_c and R, as proposed by Clark are related in that R is determined at the point of inflection on the recession side of the hydrograph. This point also defines t_c (Figure 5.1). In practice, variability from storm to storm and uncertainties in timing of recorded data may preclude reliable determination of t_c and R by direct measurement of the inflection point and the slope of the curve. In HEC-1 the interrelationship can be implicitly accounted for by estimating the parameters (t_c + R) and R/(t_c + R), rather than seeking to estimate t_c and R directly. Of course, when the "best" estimates of (t_c + R) and R / (t_c + R) are determined, values of t_c and R can be found by simple arithmetic.

5.3 Loss Rate Analysis

HEC-1 models the precipitation loss or infiltration process with 1) functions that compute loss rates using initial and uniform losses, or 2) functions that relate loss rates to rainfall and snowmelt intensity and to accumulated loss. The second category includes the exponential loss-rate (shown in Figure 5.4), the Soil Conservation Service (SCS), and the Holtan loss rate functions.



5.4. Loss Rate Function Used in HEC-1

In most cases the initial and uniform loss rate function or the SCS Curve Number procedure are used for basin loss computations. The more complicated exponential loss rate function is difficult to relate to basin characteristics, making it hard to establish regional relationships for the terms in the function. This is especially true when many subbasins are used in the watershed model.

The parameters for the initial and uniform loss rate function, the SCS Curve Number procedure, the Holton loss rate function, and the exponential loss rate function can be estimated from measured rainfall events using the optimization procedures of HEC-1 as described in the HEC-1 Users Manual (HEC, 1981). The following paragraphs cover the parameter estimation process using the exponential loss rate function; this function was used in the case studies described later in this report. Similar procedures can be used to estimate parameters for the other loss rate functions.

The function is expressed mathematically as follows:

5.4. Exponential Loss Rate Function Used in HEC-1

$$ALOSS = (AK + DLTK)\ (RAIN)^{ERAIN} \quad (5.1)$$
$$AK = STRKR/RTIOL)^{0.1\ CUML} \quad (5.2)$$
$$DLTK = 0.2\ DLTKR\ \ (1-(CUML/DLTKR))^{2} \quad (5.3)$$

where:

ALOSS = loss rate in inches (mm) per hour
AK = basic loss coefficient
DLTK = incremental loss coefficient
RAIN = rainfall intensity in inches (mm) per hour
ERAIN = exponential of the rainfall intensity
STRKR = basic loss index for start of storm in inches (mm) per hour
RTIOL = ratio of the loss coefficient (AK) to that after 10 inches (254 mm) of additional accumulated loss occurs
CUML = accumulated loss in inches (mm)
DLTKR = incremental loss index

When snow is melting, initial losses are set to zero, and the loss rate function becomes:

$$ALOSS = AK\ (RAIN + SNWMT)^{ERAIN} \quad (5.4)$$
$$AK = STRKS/(RTIOK)^{0.1\ CUML} \quad (5.5)$$

where

SNWMT = snowmelt in inches (mm) per hour
STRKS = basic loss coefficient for snowmelt in inches (mm) per hour
RTIOK = similar to RTIOL for snowmelt conditions

Snowmelt is determined using the following equation:

$$SNWMT = COEF\ (TMPR - FRZTP) \quad (5.6)$$

where

SNWMT = melt in inches (mm) per day in the elevation zone
TMPR = air temperature in ^{o}F or ^{o}C lapsed to the midpoint of the elevation zone
FRZTP = temperature in ^{o}F or ^{o}C at which snow melts
COEF = melt coefficient in inches (mm) per degree-day (^{o}F or ^{o}C)

Energy budget equations can be employed to compute melt during rain or melt during rainfree periods. Alternative versions of these functions are used for computation of losses during long-duration storms. In the loss-rate and snowmelt equations, the following parameters must be determined by calibration: STRKR, RTIOL, DLTKR, ERAIN, COEF, STRKS, RTIOK, FRZTP.

5.4 Parameter Estimation Technique

If HEC-1 were a perfect model of watershed hydrology, and if total precipitation and total direct runoff could be measured accurately, the parameters of the precipitation-runoff transformation functions for a particular storm event could be determined directly by inverse solution of the transformation equations. These conditions are not satisfied in reality, and the inverse solution of the equations is difficult. Thus the parameters cannot be determined directly. Instead, the parameters are found by selection of those values that yield the "best" reproduction of some measured runoff event with the available measured precipitation data and the available model. This parameter selection could be accomplished by a systematic trial-and-error procedure: first, parameter values can be selected; next, the model can be exercised with these values; and then, the resulting runoff hydrograph can be compared with the observed hydrograph. If the "fit" is less than satisfactory, different parameter values can be selected, and the entire process can be repeated.

An alternative to the trial-and-error approach to parameter selection is the automatic calibration approach, in which the tasks necessary for calibration are automated. Automatic calibration requires selection of an explicit index of the acceptability of alternative parameter estimates, definition of the range of feasible values of the parameters, and development of some technique for correcting the parameter estimates until the "best" estimates are determined. Thus the parameter estimation problem can be classified as an optimization problem: there is an objective function for which an optimal value is sought, subject to certain constraints on the decision variables (the parameters). Program HEC-1 incluoes the capability to solve this optimization problem, thereby automatically determining oarameter estimates that are optimal from the standpoint of the program procedures. The optimization calculations are made for a single basin only by HEC-1.

Objective Function. The objective function of the parameter estimation optimization problem must define the differences between the runoff hydrograph (computed with any parameter estimates) and the recorded runoff hydrograph. This difference will presumably be at a minimum for the optimal parameter estimates. HEC-1 employs the following function as an index of the errors:

$$STDER = \sqrt{\sum_{i=1}^{N} [(QOBS_i - QCOMP_i)^2 * WT_i]} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (5.7)$$

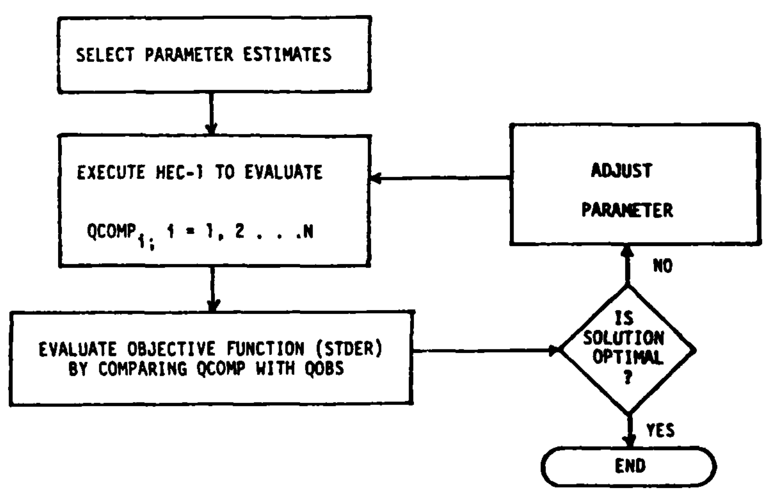
where STDER = the error index; $QOBS_i$ = observed runoff hydrograph ordinate for period i; $QCOMP_i$ = the computed runoff hydrograph ordinate for period i, computed by HEC-1 with the current parameter estimates; N = total number of hydrograph ordinates; WT_i = a weight for the hydrograph ordinate. The weight, WT_i, is defined as follows:

$$WT_i = \frac{QOBS_i + QAVE}{2 * QAVE} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (5.8)$$

where QAVE = the average computed discharge. By biasing the objective function, this weighting function emphasizes accurate reproduction of peak flows rather than low flow. Since any errors for discharge ordinates that exceed the average discharge will be weighted more heavily, the optimization scheme should focus on reduction of these errors.

The sequence of events followed for evaluation of the objective function is illustrated conceptually in Figure 5.5. The goal of the automatic calibration exercise is to select those parameter values that will yield the minimum value of STDER.

Constraints. The range of feasible values of the parameters is bounded because of physical limitations on the values that the various unit hydrograph, loss-rate, snowmelt, and streamflow routing parameters may have. Also, numerical limitations are imposed by the mathematical functions employed in modeling watershed behavior. In addition to bounds on the maximum and minimum values of certain parameters, the interaction of certain parameters is also restricted because of physical or numerical limitations. These constraints are summarized in Table 5.1. The constraints shown here are limited to those imposed explicitly in the program. Additional constraints may be appropriate in certain circumstances; however, these must be imposed externally to the program, and the user must decide whether to accept or to reject a given parameter set on the basis of engineering judgement.



5.5. Sequence of Events in Evaluation of Objective Function

TABLE 5.1

CONSTRAINTS ON HEC-1 UNIT HYDROGRAPH AND LOSS-RATE PARAMETERS

Clark Unit Graph Parameters:

$TC+R \geq 1.03\ \Delta t/(1-R/(TC+R))$

$R \geq .52\ \Delta t$

Δt = Computation Interval

Loss Rate Parameters

Exponential	Uniform
ERAIN $\leq$ 1.0	STRTL $\geq$ 0
RTIOL $\geq$ 1.0	CNSTL $\geq$ 0

Snowmelt

RTIOK $\geq$ 1.0

$-1.1\overline{1}°C \leq$ FRZTP $\leq$ 3.33°C

Optimization Technique. Addition of constraints to the parameter estimation optimization problem has a significant impact on the ease with which the problem can be solved. Whereas an unconstrained optimization problem can frequently be solved adequately with relatively unsophisticated techniques, efficient solution of the constrained optimization problem requires application of techniques that can locate an optimum while simultaneously satisfying all constraints.

The constrained optimization technique employed in HEC-1 is a univariate search technique. Application of such a technique permits use of the simulation capabilities of HEC-1 in a traditional manner and does not require development of analytical derivatives. Steps used by HEC-1 in application of this technique, shown in Figure 5.5, are as follows:

1. Initial values for parameters not assigned by the program user, are given program-assigned default values. The default parameter values are shown in Table 5.2. The parameter definitions are given in Section 5.3, with the exception of STRTL, initial loss, and CNSTL, constant loss rate, which are used with the initial and uniform loss rate method.

2. The response of the watershed is simulated with the initial parameter estimates, and the value of the objective function is computed by comparison of the ordinates of the computed and observed runoff hydrographs.

3. The parameter that is to be estimated is decreased by one percent and then by two percent, and the system response is evaluated for each change. The objective function is evaluated in each case. This gives three separate system evaluations at equally-spaced values of one parameter with all other parameters held constant.

4. The "best" value of each parameter is estimated independently by a Newton-Raphson procedure*.

5. Steps 3 and 4 are repeated for each of the other parameters to be estimated following the order in which they are listed in Table 5.2. This yields a second approximation of all parameters.

6. Step 4 is repeated for the parameter that most reduced the value of the objective function in its last change until no single change in any parameter yields a reduction of the objective function of more than one percent.

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* This procedure, which is also known as Newton's method, is described in detail in the text by Himmelblau (1972), p. 73ff. The Newton-Raphson scheme employed for estimating the "best" value of each parameter in Step 4 is based on the concept that the optimum of the objective function occurs at a root of the first partial derivative of the function with respect to each of the parameters.

TABLE 5.2

PROGRAM HEC-1 DEFAULT INITIAL UNIT HYDROGRAPH AND
LOSS-RATE PARAMETER ESTIMATES

Parameter Name	Initial Value
TC+R	$(TAREA)^{1/2}$
R/(TC+R)	0.5
COEF	0.07
STRKR	0.2
STRKS	0.2
RTIOK	2.00
ERAIN	0.5
FRZTP	32.00^{o}F
DLTKR	0.5
RTIOL	2.00
STRTL	1.00
CNSTL	0.1

*TAREA = Drainage area, in square miles

7. One more complete search of all parameters is made.

8. Step 6 is repeated, and the final parameter estimates are designated as optimal.

5.5 Application of Automatic Calibration Capability to the Rainfall-Runoff Process

Because the data available for precipitation-runoff analysis vary in quantity and form, the exact sequence of steps in application of the automatic calibration capability of HEC-1 will vary from study to study. Other sections of this report propose techniques for selecting stream gages in the study region that are representative of conditions for the basin and for developing precipitation and runoff data for use in model calibration. A proposed sequence of steps for the engineer or hydrologist to use in

determining the unit hydrograph and loss-rate parameters for the representative gaged areas is as follows:

1. Determine, for each selected storm for each gage, the recession flow for antecedent runoff (STRTQ), the discharge at which recession flow begins (QRCSN), and the recession coefficient (RTIOR) that is the ratio of flow at some selected time to the flow one hour later. The HEC-1 Users Manual (HEC, 1981) suggests techniques for estimating these parameters.

2. For each storm at each gage, determine the optimal estimates of all unknown unit hydrograph and loss-rate parameters using the automatic calibration feature of HEC-1. If the temporal and spatial distribution of precipitation is not well defined, an initial loss followed by a uniform loss rate can be used. Omit steps 3 to 7 if the initial and uniform loss rate function is used.

3. If ERAIN is to be estimated, select a regional value based on analysis of the results of Step 2 for all storms for the representative gages.

4. Repeat Step 2 with ERAIN fixed at the selected value. Select next an appropriate regional value of RTIOL, if RTIOL is unknown.

5. Repeat Step 2 with ERAIN and RTIOL fixed. Select a value of STRKR for each storm, and if desired, determine a regional value. If values for adjacent basins have been determined, check the selected value for regional consistency.

6. With ERAIN, RTIOL, and STRKR fixed, repeat Step 2 to compute TC, R, and DLTKR. DLTKR and STRKR can be generalized if desired, although the parameter DLTKR is considered event-dependent.

7. With DLTKR fixed, optimize the values of TC+R and R/(TC+R). Select appropriate values of TC+R for each gage. In order to determine TC and R, an average value of R/(TC+R) is typically selected for the

region. This value may also be appropriate for use in the ungaged basin.

8. Once all parameters have been selected, these should be verified by simulating the response of the gaged basins to other events for which precipitation and runoff records are available. If the simulated runoff hydrographs do not "match" the observed hydrograph, the calibration procedure must be repeated. The parameters should not be arbitrarily adjusted to yield satisfactory reproduction of hydrographs at this step.

9. When a satisfactory set of parameters is selected the regionalization procedures described in Chapter 6 of this report can be used to transfer the necessary information to the ungaged basin.

5.6 Streamflow Routing Optimization

In addition to the capability for modeling the precipitation-runoff processes in a basin, HEC-1 also includes the capability for modeling the movement of water through the streams and channels of the basin. So-called "hydrologic" routing techniques are used. These techniques include the modified-Puls method, the Muskingum method, the Working R and D method, the Straddle-Stagger method, the Tatum method, the Multiple Storage method, and the Kinematic Wave method. Each of these techniques is described in detail in the HEC-1 Users Manual (HEC, 1981). The kinematic wave method is also described in Chapter 7 of this report. As with the model of the rainfall-runoff process, the routing parameters must be determined from calibration; that is, appropriate values must be determined for the parameters of the equations used to model the process.

The automatic calibration capability of HEC-1 can be used to determine optimal estimates of certain parameters of the streamflow routing equations. These parameters include the following:

1. Number of routing steps to be used for routing by Tatum method, Muskingum method, or modified-Puls method (NSTPS).

2. Number of ordinates to be averaged in the Straddle-Stagger routing (NSTDL), and number of intervals the hydrograph is to be lagged (LAG) in the Straddle-Stagger routing.

3. K coefficient (AMSKK) and X coefficient (X) of Muskingum routing function.

Any additional parameters for the routing functions must be supplied by the user. This may require use of calibration techniques external to the HEC-1 optimization. For example, a storage-discharge relationship is required for application of the modified-Puls technique. This relationship cannot be determined by the automatic calibration procedure of HEC-1, but methods of developing such a relationship are discussed later in this section.

The technique used by HEC-1 for estimating the parameters of the various streamflow routing methods is similar to the technique used for estimating the parameters of the unit hydrograph and of the loss-rate functions. The steps of this calibration technique are as follows:

1. Initial parameter estimates are assigned, either by the user or by the program. The parameter values assigned by the program are shown in Table 5.3. The variable TRHR is the computation interval in hours.

2. The movement through the stream system of a specified inflow hydrograph is simulated with the parameter estimates.

3. The simulated outflow hydrograph is compared with a specified outflow hydrograph, using the same objective function that is used for determining optimal estimates of the unit hydrograph and loss-rate parameters.*

* This optimization process may not produce useful results because knowledge of local inflows is also required. The program requires a time pattern which is used to distribute the intervening-area runoff calculated by the program. The intervening-area runoff is added to the routed hydrograph flows prior to evaluation of the objective function. The required knowledge of the time distribution of intervening-area runoff may make it impossible to get good results from the optimization procedure, especially if intervening-area runoff is a significant percentage of the total runoff. "Optimal" estimates are usually very sensitive to the assumptions made regarding intervening-area runoff.

TABLE 5.3
PROGRAM HEC-1 DEFAULT INITIAL ROUTING PARAMETER ESTIMATES

Parameter Name	Initial Value
NSTPS	1
NSTDL	1
LAG	1
AMSKK	TRHR
X	0.2

4. Each parameter is optimized in the same manner as the unit hydrograph and loss rate parameters. Derivatives are estimated with finite difference approximations.

5. The optimal estimates of integer parameters, such as the number of routing steps to be used in the Tatum method, are determined by testing successive values and selecting as optimal the value preceding the first one that causes the objective function to increase.

Alternatives to the above procedures for estimating routing parameters are:

1) A storage-outflow relationship for the modified-Puls routing technique can be developed from steady-flow water surface profile computations. These computations can be done with the HEC-1 normal depth option or with computer program HEC-2 (HEC, 1979). This approach requires collection (or estimation) of stream cross section data, channel roughness characteristics, and streambed slopes necessary for computation of the water surface profiles.

2) The techniques developed for HEC by Slocum and Dandekar (1975)provide an alternative procedure for optimization of routing parameters for the modified Puls and Muskingum routing methods. A computer program - OPROUT (HEC, 1982) - is available to permit the parameter estimates to be developed outside of HEC-1. In this procedure the local inflows are computed by subtracting the ordinates of the routed hydrograph from the observed

downstream hydrograph. In nearly all cases local inflows are not known. However, it is reasonable in nearly all situations to assume that none of the local flow values will be negative. The procedure used by OPROUT employs an objective function which minimizes the sum of negative local flow in a given routing reach through definition of the storage-outflow relationship. The routing parameters which meet this objective are considered optimum.

5.7 Typical Values of Model Parameters

Table 5.4 gives values considered to be approximately the upper and lower bounds expected in practice for the various parameters used by HEC-1. In certain regions, stricter bounds may be appropriate; these can be established on the basis of experience with application of HEC-1.

TABLE 5.4

BOUNDS ON HEC-1 PARAMETERS

UNIT HYDROGRAPH AND LOSS-RATE PARAMETERS

Parameter	Lower Bound	Upper Bound
TC+R	1.5*TRHR	100
R/(TC+R)	1/3	1
STRKR	0	5
ERAIN	0	1
FRZTP	32	38
DLTKR	0	5
RTIOL	1	10

STREAMFLOW ROUTING PARAMETERS

Parameter	Lower Bound	Upper Bound
NSTPS	1	10
NSTDL	1	10
LAG	1	10
AMSKK	0	TRHR
X	0	0.5

CHAPTER 6

REGIONALIZATION OF UNIT HYDROGRAPH AND LOSS RATE PARAMETERS

6.1 Introduction

The purpose of a regional study is to develop unit hydrograph coefficients and loss rate data for basins for which no gaged hydrograph data are available. The study area must be large enough to obtain a good sampling of gaged basins. Gaged basins do not have to be within the same watershed as the ungaged sites, but all basins must be in a region that is hydrologically and meteorologically similar.

As discussed earlier, unit hydrographs are usually developed for ungaged basins in two steps. First, an equation or procedure is devised that will allow a unit hydrograph to be computed. Second, parameters of the equation or procedure are related to definable basin characteristics. The procedure used most generally involves deriving parameters that describe the unit hydrograph, and then transferring these parameters with adjustments to compensate for the geographic differences between the gaged and ungaged basins.

6.2 Basic Steps in a Regional Study

The following steps are generally followed in executing a regional study to develop loss rate and unit hydrograph relationships for a basin.

(1) Collect precipitation and runoff information in region. Data requirements for each gaged area are:

a. Precipitation data for each flood.
b. Hydrograph records for each flood.
c. Type of unit hydrograph coefficients to be derived.
d. Type of loss rate to be derived.
e. Base flow characteristics.

(2) Perform an HEC-1 rainfall-runoff analysis using subbasins above gaged locations. HEC-1 is used to estimate the unit hydrograph and loss rate parameters for the gaged drainage areas. A basin model is developed for each gaged basin for this analysis. As mentioned above, only a single subbasin can be used for optimization using HEC-1, however.

(3) Correlate unit hydrograph and loss-rate parameters with basin characteristics and develop generalized relationships. HEC-1 model results for each gaged basin are analyzed by tabulating the unit hydrograph and loss-rate data. The data are then reviewed for consistency, and an average value is chosen for each unit hydrograph and loss-rate parameter.

(4) Divide basin into subbasins. By knowing basin characteristics in the ungaged subbasins, solve for parameters using relationships developed in Step 3 above.

(5) Perform HEC-1 analysis using gaged and ungaged subbasins. If the reconstituted runoff volumes at downstream gages are significantly different from observed hydrographs, adjust the unit hydrograph, loss-rate, and routing parameters.

6.3 Definition and Description of Regionalization Parameters

The basin parameters considered most important in affecting basin hydrologic response are: drainage area, stream slope, channel length, and percent of impervious area. The following definitions of these parameters are used to provide standardized terminology in the discussions that follow:

DA = drainage area

S = equivalent stream slope of the longest watercourse

L = length along the longest watercourse from the outflow point of the designated subbasin to the upper limit to the watershed boundary.

L_{ca} = watercourse length from the outflow point to a point on the stream nearest the centroid of the basin.

I = index of impervious cover in percent of total land area.

These basin characteristics (and any others that may be influential in affecting basin behavior) are used with the unit hydrograph parameters discussed in Chapter 5 (Clark's TC and R, or Snyder's TP and CP), or the parameters describing the loss-rate function and runoff hydrograph (QRSCN, STRTQ, RTIOR, DLTKR, STRKR, RTIOL, ERAIN), either individually or in combination.

Example. If Clark's unit hydrograph method is being used, the above parameters can be correlated with TC and R either graphically or numerically (using multiple regression techniques in the latter case). The following are examples of parameters which could be used (some of which are combinations of the individual parameters):

(1) DA
(2) S
(3) $L/\sqrt{S}$
(4) $L L_{ca} / \sqrt{S}$
(5) $L\sqrt{DA/S}$

In the graphical procedure TC+R is plotted against drainage area, slope, channel length, or some combination of these parameters, and a best-fit line is drawn through the points. If a multiple regression analysis is used to obtain an equation for TC+R, the same parameters or parameter groups can be used. Residuals, which are the differences between the filtered or calculated values and the observed values, can be obtained from the multiple regression results. (Residuals are also known as remainders, discrepancies, and differences.) These residuals can be plotted on a study area map and lines of equal residual can be drawn. TC+R determined from the regression equation can be adjusted by using the residual shown on the map for the ungaged basin(s) being analyzed.

Recession and Base Flow Parameters. The next step is to define the input variables that describe the base flow and recession portion of the hydrograph: STRTQ, QRSCN, and RTIOR. The initial and base flow parameters can be related to basin characteristics in the following ways:

(1) STRTQ can be related to characteristics that are indicative of antecedent moisture conditions and size of the basin. Drainage area is often used.

(2) QRSCN can be related to peak discharge (Q_{peak}), precipitation intensity, drainage area, or other parameters. It should in general be no greater than 25 percent of the peak and is typically between 3 and 10 percent.

(3) RTIOR can be related to Q_{peak}, precipitation intensity, drainage area, etc. RTIOR is often assumed to be constant throughout a region.

The loss-rate function is often selected at this point. If the temporal and spatial distributions of precipitation are not well defined, an initial loss followed by a uniform loss rate is frequently used. The next step is to run the HEC-1 model with the runoff parameters developed. HEC-1 will estimate values of TC+R, R/(TC+R), STRKR, and DLTKR, if observed hydrographs are available with representative rainfall data. (Values of TC, R, TP, and CP based on calibration results are also given as program output).

Analysis of Loss Rate Parameters DLTKR and STRKR. When QRSCN, STRTQ, RTIOR, RTIOL, ERAIN, TC and R, or TP and CP were fixed in earlier steps, errors associated with estimating basin average precipitation, the losses, or other parameters were lumped together in the terms DLTKR and STRKR. As a consequence of this lumping, it is very difficult to relate STRKR to basin characteristics. The values of DLTKR and STRKR, which are determined for a particular storm event at a gage, are often used without modification at the ungaged sites. If estimates of historical peak discharges exist at ungaged sites, then STRKR and DLTKR can be adjusted so the reconstituted peak equals the historical peak. As a check of the method, a HEC-1 simulation should be made for gaged and ungaged regions using the parameters developed in the above steps. Adjustments can be made if calculated and observed downstream hydrograph volumes are different.

6.4 Basic Techniques for Regression Analysis

Correlation techniques available for regional analysis include the following: (1) graphical correlation, (2) simple linear regression, and (3) multiple linear regression. A nonlinear relationship can sometimes be transformed into a linear relationship (for example, by a logarithmic transformation), and linear regression can be applied to the transformed values.

Graphical Correlation. The various regionalization parameters are plotted as a function of each other. Usually logarithmic graph paper is used, and a best fit line is drawn by eye. Other information, such as knowledge of the approximate slope of the curve or limits on some of the parameters, can often be used to aid in positioning the curves. An example is shown in Figure 6.1, where values of t_p computed by optimization runs of HEC-1 are plotted as a function of LL_{ca}. It was assumed that the slope of these curves should be the same as those established in other studies (i.e., the slope is equal to 0.30).

Linear Regression. As in the graphical correlation case, it is assumed that the variables (or their logarithms) are related to each other by the equation for a straight line. The regression equation is:

$$Y = a + b X \quad \text{(6.1)}$$

where Y = the dependent variable, or its logarithm

a = regression constant,

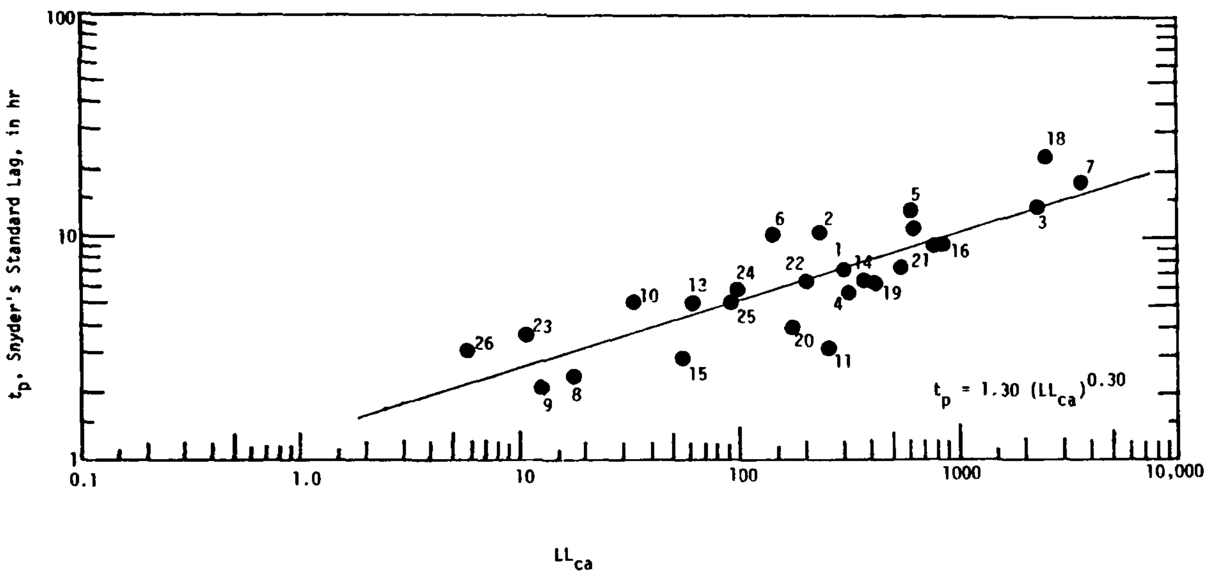
b = regression coefficient, and

X = the independent variable, or its logarithm

Many programs are available for determining values of a and b that give a least-squares best-fit to a given set of data. These include programs for hand-held or desk-top calculators. Hand calculations can be made also if the data sets are not large.

	Gage Location	t_p (hours)	DA (sq mi)
1	Pohopoco Creek near Parryville	7.1	109.0
2	Aquashicola Creek at Palmerton	10.2	76.7
3	Lehigh River at Walnutport	13.4	889.0
4	Little Lehigh River near Allentown	5.5	80.8
5	Jordan Creek at Allentown	12.3	75.8
6	Monocacy Creek at Bethlehem	10.2	44.5
7	Lehigh River at Bethlehem	17.0	1,279.0
8	Saucon Creek at Lanark	2.4	12.0
9	So. Branch Saucon Cr. at Friedensville	2.0	10.6
10	Saucon Creek at Friedensville	5.0	26.6
11	Tohickon Creek near Pipersville	3.1	97.4
12	Neshaminy Creek near Langhorne	10.2	210.0
13	Schuylkill River at Pottsville	5.0	53.4

	Gage Location	t_p (hours)	DA (sq mi)
14	Schuylkill River at Landingville	6.0	133.0
15	Little Schuylkill River at Tamaqua	2.9	42.9
16	Schuylkill River at Berne	9.2	355.0
17	Tulpehocken Creek at Reading	7.1	211.0
18	Schuylkill River at Pottstown	22.8	1,147.0
19	Perkiomen Creek at Graterford	6.0	279.0
20	Ridley Creek at Moylan	3.9	31.9
21	Brandywine Creek at Chadds Ford	9.2	287.0
22	Chester Creek at U.S.G.S. gage	6.4	61.1
23	Maiden Creek Tributary at Lenhartsville	3.7	7.5
24	French Creek near Phoenixville	5.7	59.1
25	Skippack Creek near Collegeville	5.1	53.7
26	Pickering Creek near Chester Springs	3.1	6.0



6.1. Graphical Correlation - t_p as a Function of LL_{ca}

Multiple Linear Regression. In general, watershed response is dependent on several watershed parameters. An equation of the following form can be used to provide a mathematical expression that involves several independent variables:

$$Y = a + b_1X_1 + b_2X_2 + \dots + b_nX_n \qquad (6.2)$$

where Y = the dependent variable,

a = regression constant,

$b_1, b_2 \dots$ = regression coefficients, and

$X_1, X_2 \dots$ = independent variables

This type of analysis generally requires a computer program, and most computer systems have multiple regression program packages. For example, HEC has a step-wise multiple regression program (702-G1-L2020) available in FORTRAN IV (HEC, 1970). Other multiple regression programs, such as the Statistical Package for Social Sciences (Nie et al., 1975), are available on most large computing systems.

When several watershed parameters are being considered, some of the proposed parameters may have little effect on the dependent variable. These parameters, of course, should be dropped from consideration, and the final expression should include only those parameters which significantly affect the result. The general rule is: the fewer variables the better. To determine which parameters should be dropped, a "step-wise regression" procedure is used.

In step-wise regression the analysis is first made with all the specified independent variables; the least significant variable is then deleted, and the analysis is repeated. In the HEC multiple regression program (HEC, 1970) the independent variables are deleted, in turn, on the basis of the adjusted partial determination coefficient (r^2), which gives the increase in unexplained variance caused by deleting that variable from the regression equation.

The HEC program also permits combining and transforming variables. New variables can be computed by combining individual input variables; for

example, a parameter $DA\sqrt{S}$ (basin area DA times the square root of the channel slope S) can be computed if the area and slope have been specified as program input. Individual variables may be transformed (by taking the square root, reciprocal, or logarithm) to make the relationships more nearly linear.

<u>Logarithmic Transformation of Nonlinear Relationships</u>. If the relationship is nonlinear, it can often be transformed into a linear relation; for example, the equation $Z = cX^dY^e$ is equivalent to $\log Z = \log c + d \log X + e \log Y$ through a logarithmic transformation. After such a transformation is made, linear regression can be used to determine the coefficients that best fit the data.

6.5 Criteria for Accepting Results of Regression Analysis

The results of the regression analysis are evaluated by looking at the statistics describing the "goodness-of-fit" of the regression equation to the data. The statistical parameters used are the coefficients of determination (both the adjusted coefficient and the unadjusted coefficient), the partial determination coefficient, and the standard error of estimate. A detailed discussion of these statistical measures is provided by Beard (1962).

The unadjusted multiple-determination coefficient (R^2), and the adjusted multiple-determination coefficient (R^2), provide a measure of the percent of variance in the dependent variable explained by the independent variable. The magnitude of these coefficients varies from 0 to 1. The closer the value is to unity, the greater the reliability of the estimate.

The partial-determination coefficient (r^2) gives a measure of the importance of an independent variable by determining the reduction in variance in the dependent variable when the variable is included with the other independent variables.

The standard error of estimate (S_e) is the standard deviation of the differences between the observed dependent values and the values computed from the regression equation in the units of the dependent variable; therefore, it must be compared with the mean and the standard deviation of that variable.

When regression analysis is used to determine equations relating the various model parameters, the standard error of estimate and the coefficient of determination are computed for each equation. Typical results of such an analysis are tabulated in Table 6.1, where values of S_e and R^2 are given for each equation. The general rule is to use R^2 and S_e as a guide and select the equation with the fewest independent variables and the best values of R^2 and S_e.

TABLE 6.1

TYPICAL RESULTS OF MULTIPLE REGRESSION ANALYSIS
FOR REGIONALIZATION OF MODEL PARAMETERS
(RAHWAY RIVER BASIN, NEW JERSEY)

	Standard Error of Estimate S_e	Correlation Coefficient R	Coefficient of Determination R^2
$TC = 26.19\ I^{-0.53} S^{-0.29}\ (DA)^{0.23}$	0.0495	0.9710	0.9428
$TC = 19.84\ I^{-0.50}\ (DA/S)^{0.26}$	0.0358	0.9849	0.9701
$TC = 8.29\ K^{-1.28}\ (DA/S)^{0.28}$ *	0.0269	0.9915	0.9831
$TC = 4.14\ (DA/S)^{0.39}$	0.1296	0.7800	0.6084
$(TC + R) = 122.64\ I^{0.42}\ S^{-0.55} (DA)^{0.09}$	0.1442	0.6844	0.4684
$(TC + R) = 15.69\ I^{-0.21} (DA/S)^{0.34}$	0.1161	0.8094	0.6552
$(TC + R) = 11.52\ K^{-0.67}\ (DA/S)^{0.33}$ *	0.1054	0.8461	0.7159
$(TC + R) = 7.98\ (DA/S)^{0.39}$	0.1093	0.8333	0.6944

* $K = 1.0 + 0.03I$

CHAPTER 7

HEC-1 KINEMATIC WAVE ROUTING TECHNIQUES FOR UNGAGED BASIN MODELING

7.1 Introduction

The purpose of this chapter is to describe in general terms how the HEC-1 kinematic-wave modeling prodedures may be applied. A discussion of kinematic wave theory as it relates to the HEC-1 procedures are given by DeVries and MacArthur (1979). The papers by Lighthill and Whitman (1955), Harley et al. (1972), and Woolhiser (1975) are recommended for general background on the kinematic wave method. The kinematic wave approach to rainfall-runoff modeling uses a very detailed analysis of the physical processes occuring on the watershed. This is in constrast to the equally valid, but less detailed, "lumped-parameter" approach used in the unit hydrograph method. In the kinematic wave approach the various physical processes of water movement over the basin surface (with attendant infiltration), flow into stream channels, and flow through the channel network are modeled using the equations of physics. Parameters such as roughness, slope, catchment lengths and areas, and stream channel dimensions are used to define the processes.

The various features of the irregular surface geometry of the basin are approximated by two types of basic elements: (1) an overland flow element, and (2) a stream or channel flow element. In the modeling process described here, one or two overland flow elements (designated as overland flow strips) are combined with one or two channel flow elements to represent a subbasin. An entire basin is modeled by linking the various subbasins together.

Because the various elements comprising the model are defined in terms of physical parameters, the model can be easily modified, and changes which represent changes in land use in the basin can be made using parameters which describe these new uses. This makes kinematic- wave-type models very useful for urban studies because the effects of increasing urbanization can be accounted for by changing the parameters describing the basin.

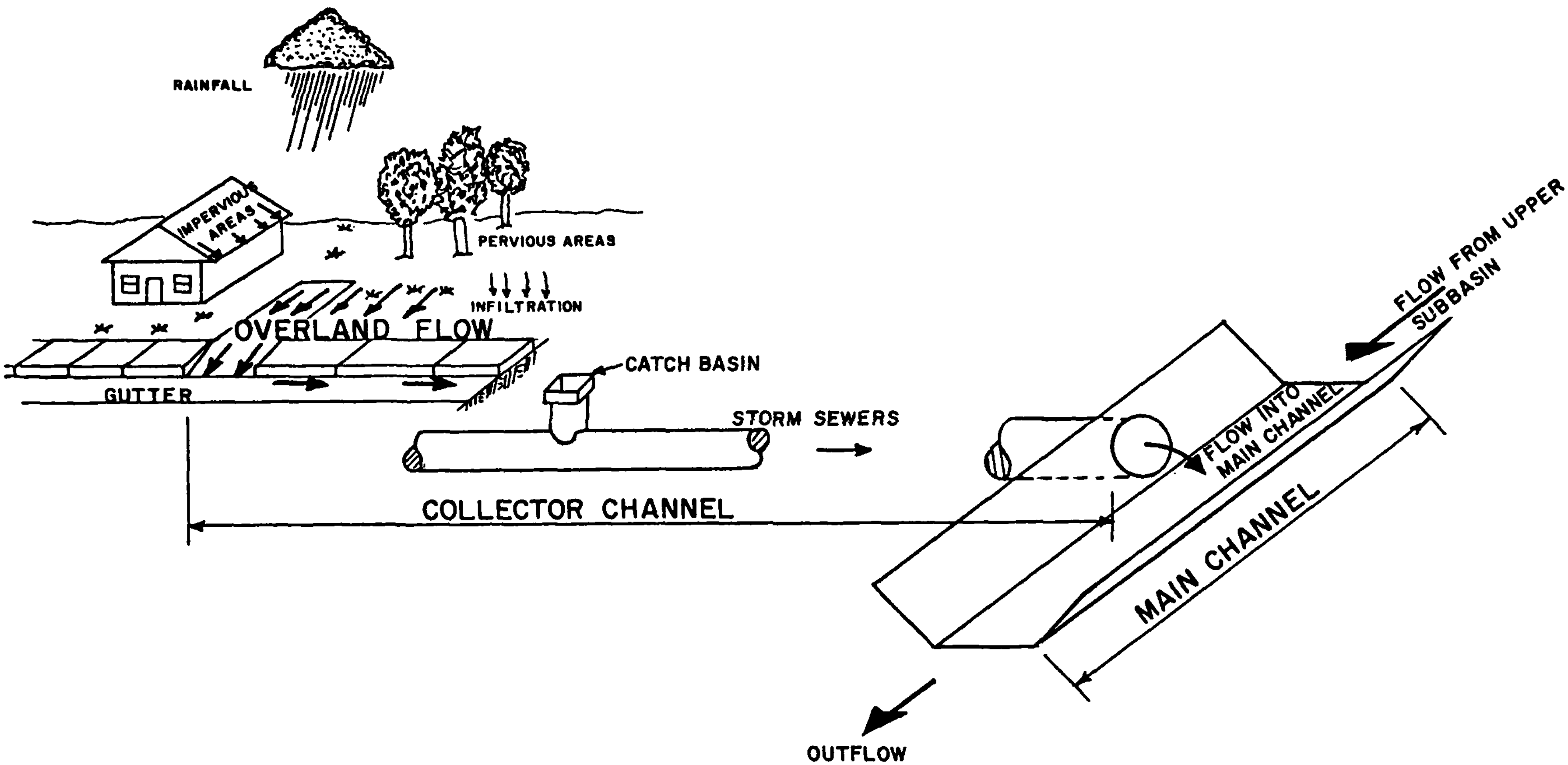
The various topics which are covered here include basin modeling procedures, a description of the elements used in kinematic wave calculations, and procedures for selecting the parameters. An example problem is presented to illustrate HEC-1 input and output data, and effects of changes to numerical values of the parameters are discussed.

7.2 Basin Modeling

The modeling process starts with a description of the topologic structure of the basin: drainage basin boundaries, stream and drainage channels, and the logical relationships between the drainage areas and the channels. The definition of the drainage boundary will depend on the objective of the study being conducted, as well as the topological character of the basin. Studies dealing with urban hydrology usually require delineation of subbasins that are smaller than 2 mi^2 in extent (about 5 km^2). Studies dealing with the effects of channel modifications may permit use of large areas; however, as the area is increased the assumptions required to apply the kinematic wave method become more tenuous.

Typical elements of an urban drainage system are shown in Figure 7.1. Rain falls on two general types of surfaces: (1) those that are essentially impervious, such as roofs, driveways, parking lots and other paved areas; and (2) pervious areas, most of which are covered with vegetation and have numerous small depressions which produce local storage of rainfall. It is assumed in the model that water initially travels over these surfaces as sheet flow; however, in a relatively short distance the water begins to collect in small streams or rivulets and the process of stream or channel flow begins. For impervious areas, the distance to the first channel (say a gutter) is typically thirty to one hundred feet, while for pervious surfaces the longest distance a drop of water must travel to reach a channel is on the order of one hundred to several hundred feet.

Water collected by the gutters usually travels no more than a few hundred feet until it enters catch basins which are connected to sewers. These sewers are typically 1.5 to 2 feet in diameter for local drains. Local drains are connected in turn to larger and larger drains which feed the main storm drain. In many areas the main storm drains are open channels or



7.1. Typical Urban Drainage Pattern

streams. In major urban areas the main storm drains are often large closed-conduit sections, but these storm drains are usually designed to flow only partially full, and therefore, the kinematic wave routing approach (which assumes open channel flow) is appropriate.

7.3 Kinematic Wave Equations

The kinematic wave equations (DeVries and MacArthur, 1979) are the continuity equation for unsteady open channel flow with lateral inflow, and Manning's equation. The continuity equation is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_o \quad (7.1)$$

where Q is the channel flow in cfs, A is the flow cross-sectional area in ft^2, q_o is the lateral inflow in cfs/ft, x is the distance along the flow path in ft, and t is time in seconds. Manning's equation is written in the form:

$$Q = \alpha A^m \quad (7.2)$$

where the kinematic wave routing coefficients, α and m, are functions of the channel or flow surface geometry. The general expression for α is:

$$\alpha = \frac{KS^{1/2}}{n} \quad (7.3)$$

where K is a constant that depends on the geometry, S is the slope, and n is a roughness coefficient. A change in either n or S will change the value of α used in the calculations. The same equations are used for both sheet flow computations (for overland flows) and for channel flow computations.

7.4 Elements Used in Kinematic Wave Calculations

The runoff process described above is idealized in HEC-1 through the use of the following flow elements: (1) one or two overland flow elements, (2) one or two collector channel elements, and (3) a main channel element. These

generally provide the necessary detail for modeling the runoff process in urban basins. Schematic drawings of these elements are shown in Figure 7.2. The elements are specified to represent typical features of the basin, and thus the parameters chosen for the individual elements should be representative of the entire subbasin. Because land use and development practices are usually very similar within a selected hydrologic unit, assigning a single value to a given parameter (such as an overland flow length for impervious areas, for example) usually gives good results.

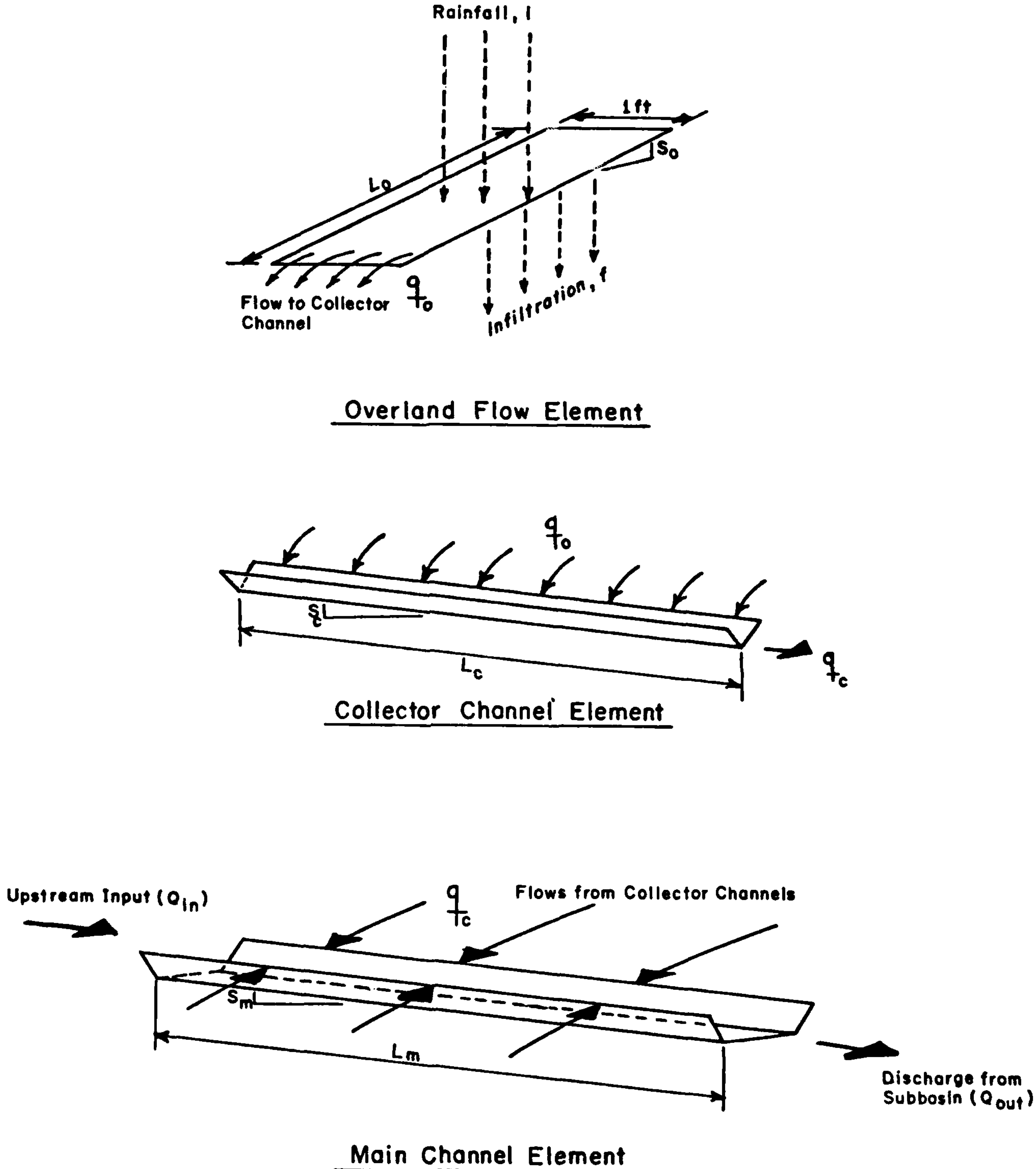
7.5 Overland Flow Elements

The basic overland flow element is simply a sloping rectangular plane upon which the rain falls. In HEC-1 it is treated as a strip of unit width (one foot or one meter wide). Some of the rainfall is lost by infiltration; the remainder runs off the lower edge of the plane into a channel. Infiltration losses may be described as varying with time or constant, and any of the previously described loss rates can be specified for each flow strip. The fraction of the element that is impervious can also be specified. The losses are computed first, and then the overland flow computations are made.

The basic kinematic-wave-analysis concept used in HEC-1 allows the use of either one or two overland flow surfaces, each discharging into a collector channel. For example, one element could represent all areas that are essentially impervious, with short lengths of flow (L_o) to the point where the flow becomes channel flow. Thus the element would represent driveways, roofs, street surfaces, etc.

The other overland flow element could then represent areas that are pervious and have higher resistance to flow, such as lawns, fields, and wooded areas. In general, the catchment flow lengths and roughness coefficients will be much greater for these areas. Again, the value of L_o to be used is the representative maximum distance for water to travel as overland flow for this type of land surface.

The user of this method should think of the overland flow strips as representing typical flow surfaces rather than actual planar surfaces, except



7.2. Elements Used in Kinematic Wave Calculations

when very small areas (such as one city lot) are being considered. It is only at these very small scales that the mean surface slope and actual area and length come close to fitting the basic theoretical concept.

Fortunately, in many natural basins and urban catchments, close examination of the full drainage system reveals that the small-scale drainage patterns are quite similar throughout the entire basin. The value of L_o appropriate for such a situation will not vary greatly over the basin. The actual values of L_o which give the correct runoff response for the basin must be verified through comparison of model output with measured data, however.

The following data are needed as input to HEC-1 to describe each overland flow strip:

1. L_o - typical overland flow length
2. S_o - representative slope
3. N - roughness coefficient
4. A_o^1 , A_o^2 - the percentages of the subbasin area which the overland flow surface represents (two possible types for each subbasin)
5. Infiltration and loss-rate parameters

7.6 Collector Channel

The collector channel element is used to model the flow in its path from the point where it first becomes channel flow to the point where it enters the main channel. The inflow to the collector channel is taken as a uniformly distributed flow along the entire length of the channel. This correctly represents the situation where overland flow runs directly into the gutter, and also provides a reasonable approximation of the flow inputs into the storm drain system from individual catch basins and tributary collector pipes which are distributed along the collector channel.

The value of the exponent m in Equation 7.2 for trapezoidal channels ranges from 4/3 when the trapezoid has a base width of zero (triangular shape) to 5/3 for a very wide rectangular shape. For a channel with a circular cross section, m is taken as 1.25.

The following data are needed as input to describe the collector channel system:

1. A_c - surface area drained by a single representative collector channel (e.g., gutter plus storm drain)
2. L_c - collector channel length (total length of gutter plus length of storm drain)
3. S_c - channel slope
4. n - Manning's roughness coefficient
5. Channel shape (either a circular section or some variant of a trapezoid can be used)
6. Pipe diameter or the trapezoid bottom width and side slope, if appropriate

7.7 Main Channel

The main channel can carry both inflows from upstream subbasins as well as flows supplied by the collector channels within the subbasin. The inflow from the collector channel is taken to be uniformly distributed along the length of the main channel. This is assumed to reasonably approximate the actual situation where the flow enters the channel from the various collectors at a number of discrete points at various spacings.

The channel routing element can also be used independently for routing a hydrograph through a channel reach. If desired, the subbasin flow can be computed separately and combined with routed flow at the subbasin outlet. Any of the routing methods available in HEC-1 can be used for channel routing (Muskingum, modified-Puls, Tatum, etc.) if desired.

The channel routing procedure requires the following data:

1. $A_{subbasin}$ - Area of subbasin
2. L_m - Channel or stream length
3. S_m - Slope
4. n - Manning's roughness coefficient
5. Channel shape (trapezoidal or circular)

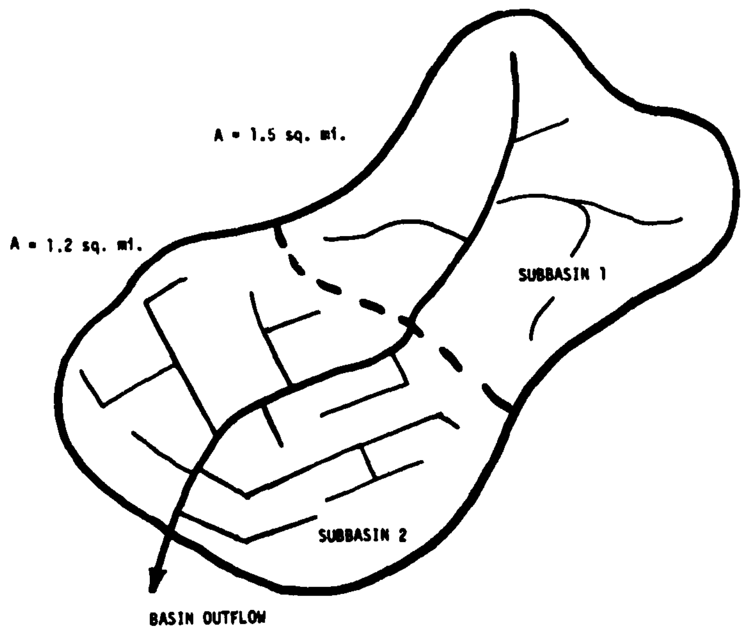
6. Channel dimensions (e.g., width, w, or diameter, D, if required, and side slopes, z)
7. The upstream hydrograph to be routed through the reach, if desired.

7.8 Example Application of Kinematic Wave Methods

The small partially-urban basin shown in Figure 7.3 is to be modeled using kinematic wave runoff and routing options of HEC-1. The hydrologic characteristics of this basin (obtained either by previous calibration or by comparison with other basins) are as follows:

Subbasin 1. The upper of the two subbasins is presently not urbanized and is primarily rolling pasture with few trees. The typical distance L_o for flow to travel to tributary stream channels is 500 feet. The overland flow roughness coefficient N is 0.4. The representative ground slope, S_o, is 0.04. The amount of impervious area is assumed to be negligible. The subbasin area A_o^1 is 1.5 square miles.

The collector or tributary channels have a slope, S_c of 0.025, and an 'n' value of 0.10, with a typical channel length, L_c, of 1,500 feet. The most representative section is a triangle. The area, A_c, contributing to a typical collector stream is 0.4 square mile.



7.3. Basin for Example Problem

The main channel is approximately triangular in cross section with side slopes, z, of 1 in 4. The mean channel slope, S_m, is 0.01 and Manning's 'n' is 0.05. Its length, L_m, is 3,500 feet.

Subbasin 2. The lower subbasin is completely urbanized, and twenty percent of the subbasin surface is impervious. In this subbasin the impervious runoff areas have the following characteristics: L_{o_1} = 50 ft, S_{o_1} = 0.06, N = 0.15. The pervious areas can be represented by the following parameters: L_{o_2} = 130 ft, S_{o_2} = 0.01, N = 0.3. the subbasin area, A_{o_2} , is 1.2 sq mi. The total basin area is 2.7 sq mi.

The collector channel system involves 1,800 ft of pipe storm drain ranging up to four feet in diameter, plus an additional three hundred feet of gutter between catch basins. A triangular section is used to represent these various channel components (the program default value which gives one to one side slopes is used). The average slope, S is 0.008, and the Manning's 'n' (which is used here to account for friction and other channel head losses) is 0.020. The area, A, contributing to the collector channel system is 0.35 sq mi.

The parameters describing this basin are given in Table 7.1, and hydrographs from the HEC-1 output are shown in Figure 7.4.

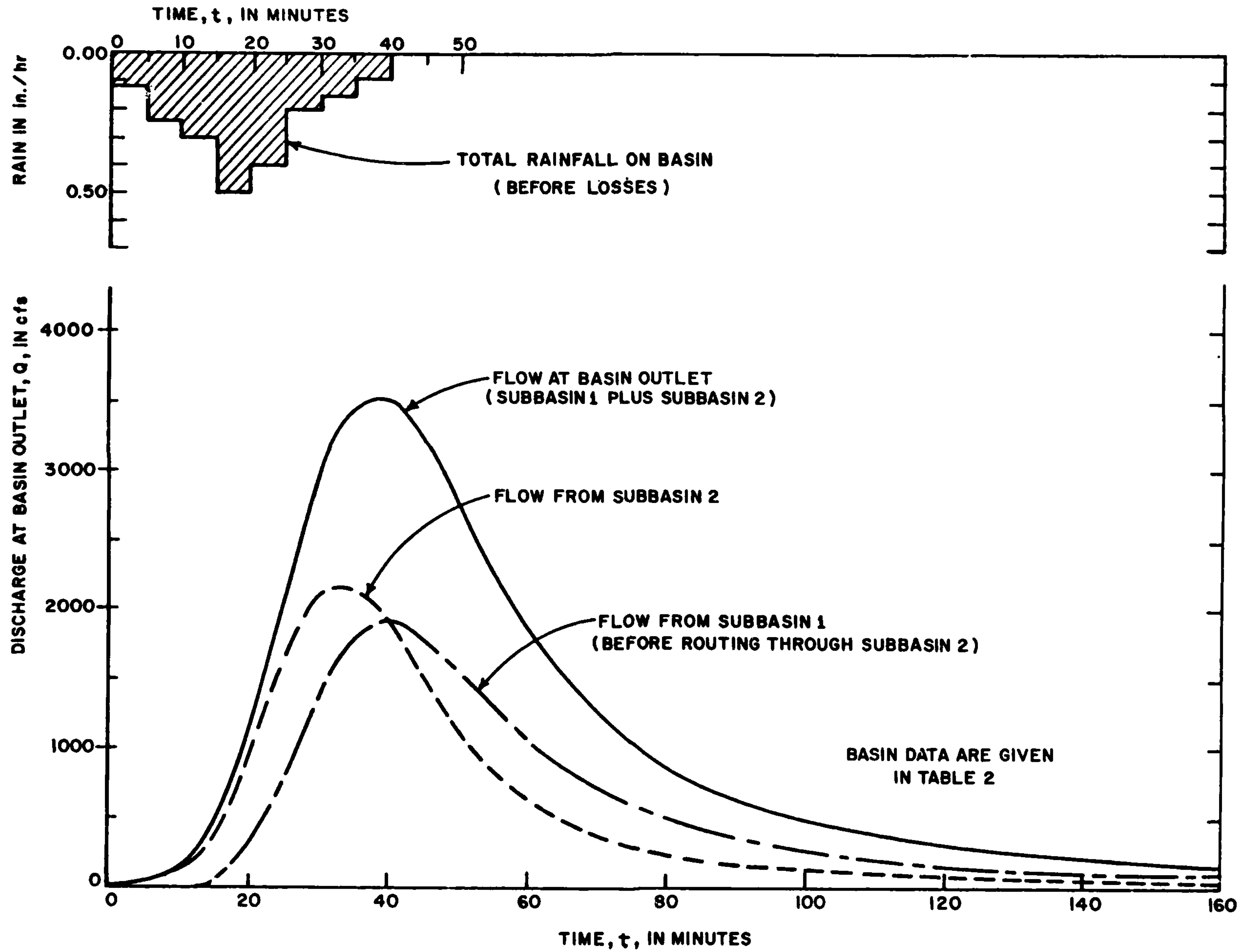
As an example of the way the program can be used to evaluate the effects of future urbanization, the following changes were made to the parameters describing Subbasin 1. Two overland flow strips were used instead of one; an impervious overland flow element with L_{o_1} = 50 feet, N = 0.3, and representing twenty percent of the subbasin area; and a pervious element with the representative flow lengths reduced to four hundred feet and with the other parameters as before.

The results from this analysis are plotted in Figure 7.5. The peak flow at the basin outlet increased from 3,514 cfs for the initial basin condition to 4,275 cfs after the urbanization changes. The peak flow occurred five minutes earlier in the fully urbanized basin.

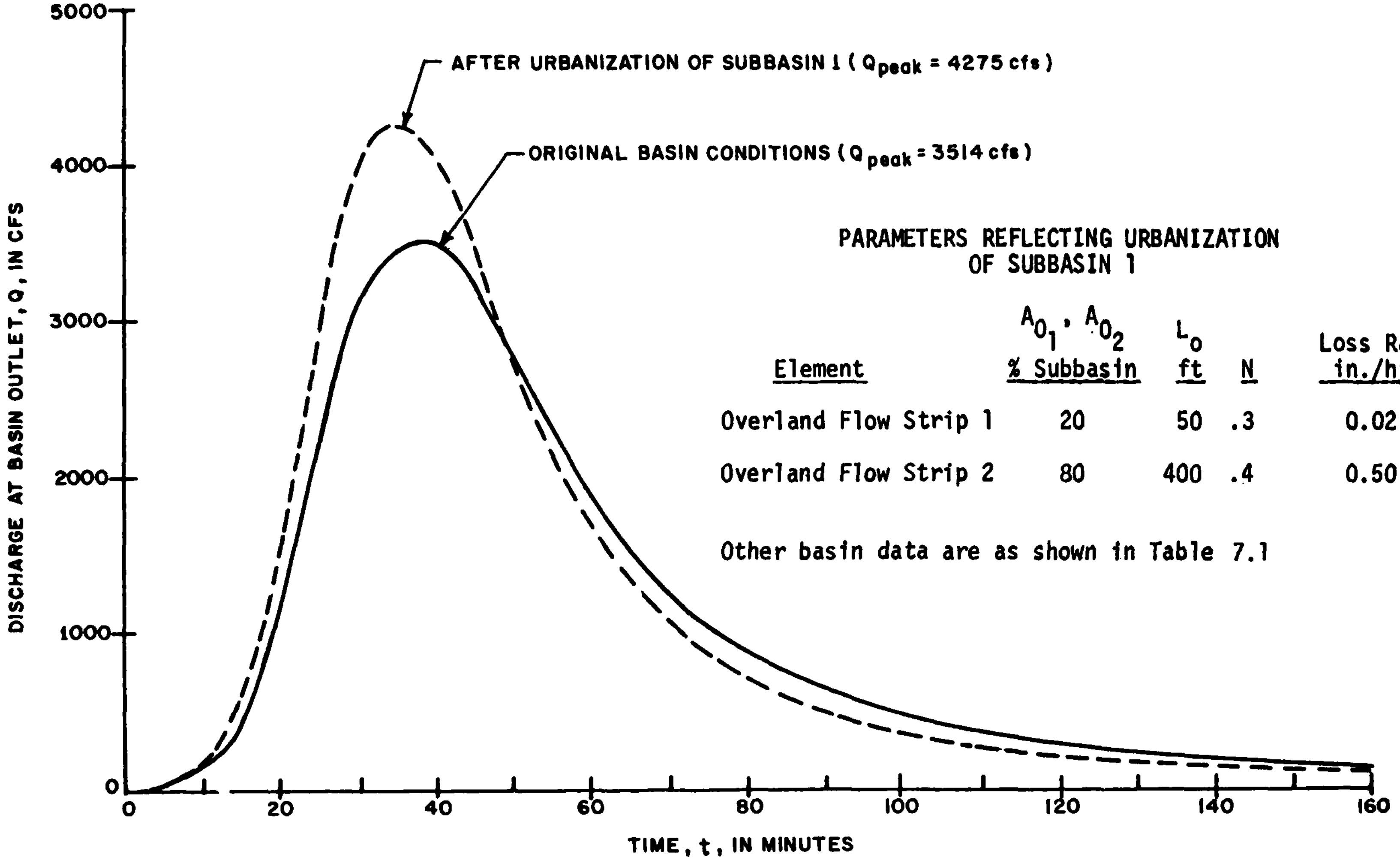
TABLE 7.1

Parameters for Kinematic Wave Example Problem

	Percent Total Subbasin Area	L ft	S	Roughness	Channel Shape	Channel Size ft	Z	Collector System Area mi^2	Loss Rate in/hr
Subbasin 1 (1.5 mi^2)									
1. Overland Flow Strip	100	500	0.04	0.40	--	--	-	--	0.50
2. Collector Channel	--	1,500	0.025	0.10	Triangle	--	1	0.40	--
3. Main Channel	--	3,500	0.010	0.05	Triangle	--	4	--	--
Subbasin 2 (1.2 mi^2)									
1. Overland Flow Strip 1	20	50	0.06	0.30	--	--	-	--	0.02
2. Overland Flow Strip 2	80	180	0.01	0.40	--	--	-	--	0.20
3. Collector Channel	--	2,100	0.008	0.020	Triangle	--	1	0.35	--
4. Main Channel	--	4,000	0.003	0.025	Trapezoid	2	2	--	--



7.4. Computed Hydrographs for Example Problem



Element	A_{0_1}, A_{0_2} % Subbasin	L_o ft	N	Loss Rate in./hr.
Overland Flow Strip 1	20	50	.3	0.02
Overland Flow Strip 2	80	400	.4	0.50

7.5. Effect of Urbanization on Basin Outflow

7.9 Summary

The material presented here provides background information for modeling hydrologic basins using kinematic wave routing with HEC-1. The example problem gives an illustration of the use of the method and shows how it can be applied to studies of the effects of urbanization on hydrologic basins. Because the kinematic wave parameters are directly related to physical properties of the watershed, the method can readily be used to develop models for ungaged basins. The major uncertainties in its use lie with the determination of roughness coefficients and flow surface slopes. It is important that the modeler have experience with similar watersheds before making estimates of parameters for ungaged areas.

The user of this method should verify the model using measured rainfall-runoff events to allow the assessment of the performance of the model with the selected parameters. Without such a check, the results should be interpreted with a great deal of caution. However, kinematic wave models have been used successfully in a large number of applications for ungaged watersheds, especially in urban settings, and when the models are properly formulated, good simulations result.

CHAPTER 8

CASE STUDY I - ANALYSIS WITH SOME DISCHARGE-FREQUENCY DATA

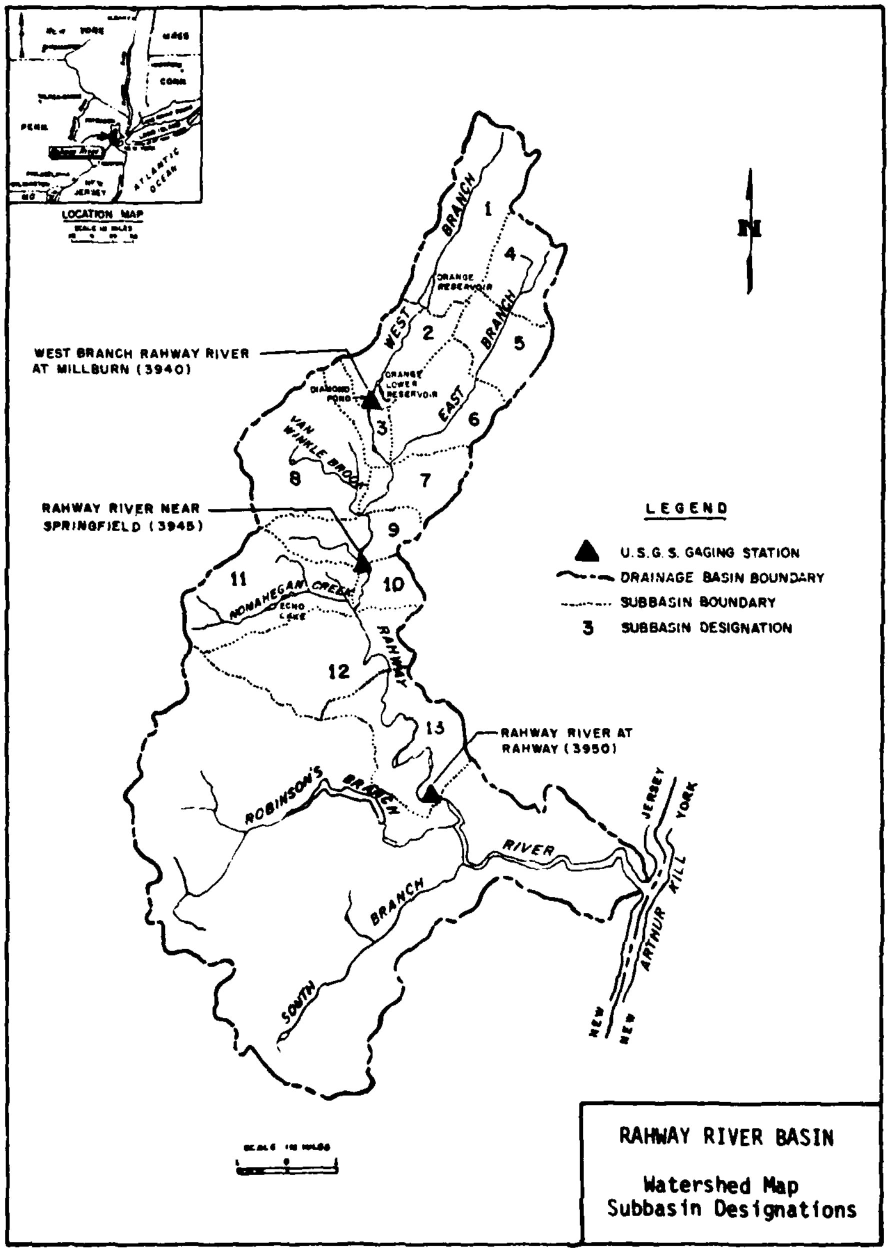
Three case studies are presented in this report to illustrate how the methods previously described are actually applied. Each provides an example dealing with a different degree of data availability. This first example discusses a study made to determine discharge-frequency relationships for the Rahway River Basin in New Jersey (HEC, 1976a). The basin is shown in Figure 8.1. There are three stream gages within the basin. At only one gage, however, was the length of record sufficient to provide an adequate discharge-frequency relationship. Discharge-frequency curves were needed at a total of 13 locations in the basin.

8.1 General Study Program

To provide the needed discharge-frequency relationships, an HEC-1 model for the basin was set up and calibrated using data from several storms. The optimization features of HEC-1 were used to develop unit hydrographs and loss rate parameters to reconstitute the observed runoff hydrographs at the gaging stations.

The basin was then subdivided into 13 subbasins related to the locations at which discharge-frequency curves are needed (index points). These are locations of existing and proposed flood control structures, major confluences, and points where there is significant change in land use, stage-damage relations, or stream-routing characteristics. Unit hydrograph parameters (t_c and R) were computed by regional techniques. The three gage locations in the Rahway basin were used to define drainage basin characteristics, and data from three nearby basins were also used.

Precipitation loss rates were estimated for the ungaged subbasins by a combination of techniques: (1) using results of the optimization runs for the gaged areas of the basin, (2) using data from hydrologically similar basins in the area, and (3) reproducing estimated peak flows at various locations in the basin using high water marks. Storage-outflow relationships for streamflow routing were determined using preliminary runs of computer program HEC-2. The full basin model was then calibrated with three storms of record.



8.1. Watershed Map - Rahway River Basin

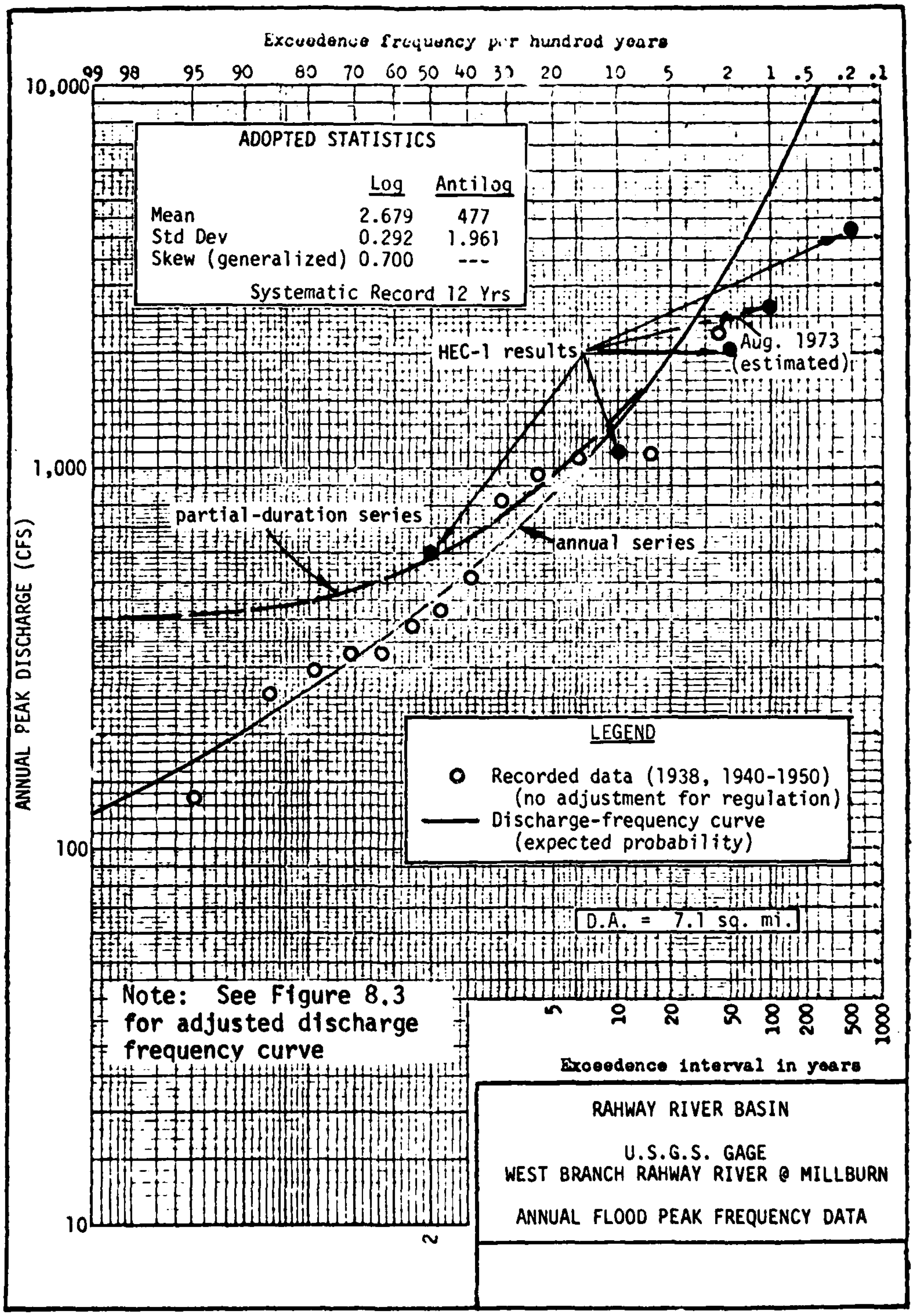
The next step in the study was to develop hypothetical rainfall data. These data were used as input to the HEC-1 model, and a discharge for each storm frequency was computed at the gage locations. The resulting discharge-frequency curve was compared with the curve computed from gage records. Next the loss rates were adjusted to make the computed results match the curve. Figure 8.2 shows the HEC-1 results as well as a plot of the recorded gage data. The recommended discharge-frequency curve is shown in Figure 8.3. The frequency curve based on HEC-1 results was compared for closeness of fit with the recommended curve, and where necessary, adjustments were made. These adjustments were then used to modify tributary curves to represent consistant runoff frequencies.

A basin model was calibrated for the area under investigation using computer program HEC-1 for selected storms of record. Using discharges computed by the HEC-1 model, river stages were computed throughout the study area for the various storm events, giving the final results desired for the study. The work performed in the study is discussed in more detail in the following paragraphs.

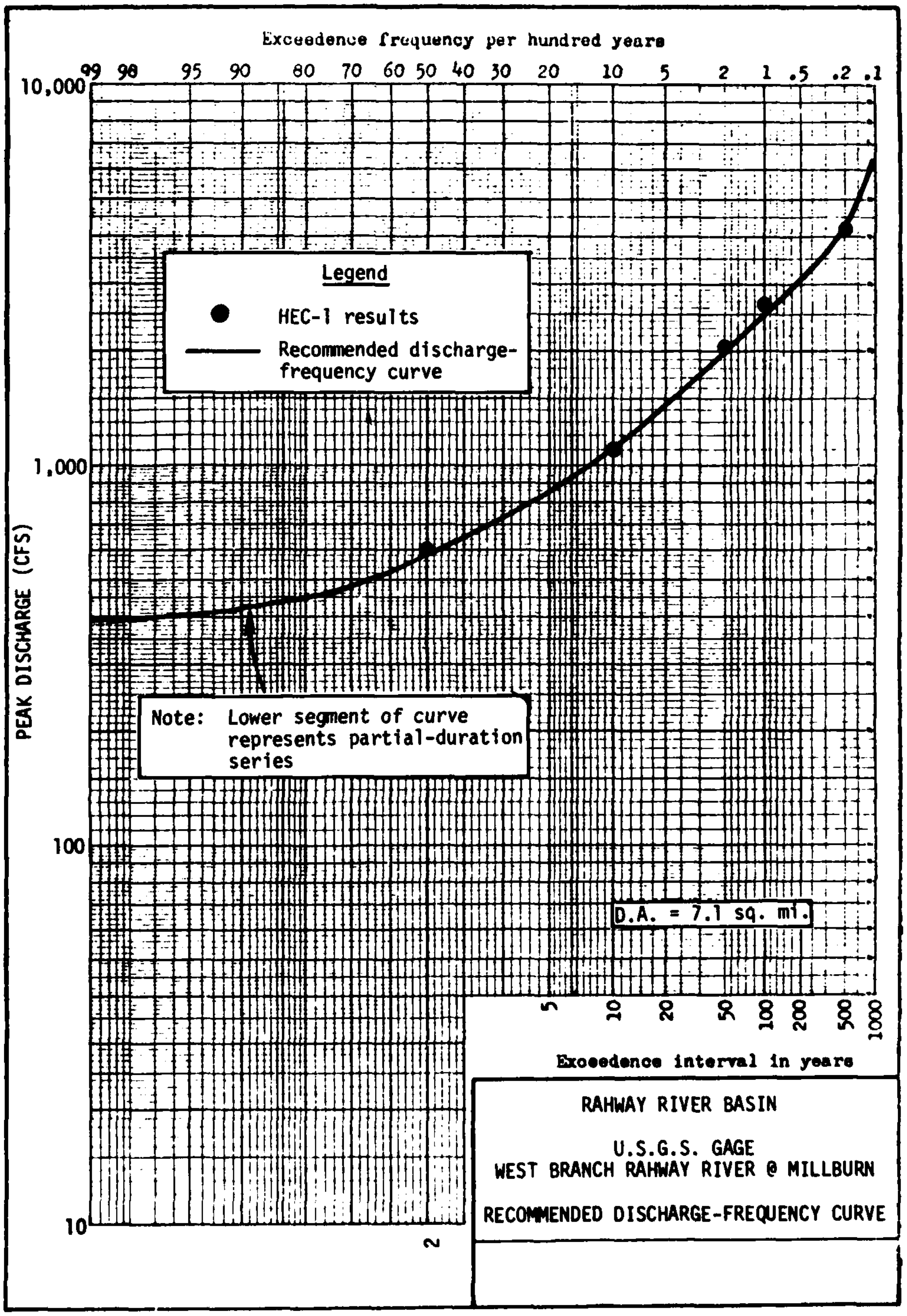
8.2 Basin Description

The Rahway River Basin is located in northeastern New Jersey within the metropolitan area of Greater New York. The basin has a total area of 81.9 square miles. The Rahway River is a tributary to the Arthur Kill, an estuary of New York Harbor. Elevations range from sea level at the confluence with the Arthur Kill to 600 ft at its most northern source. The total length of the longest watercourse is about 25 mi. Stream slopes are very small in the lower reaches and range from about 10 ft per mi in the middle reaches to 300 ft per mi in the uppermost reaches.

The East and West Branches of the Rahway River (see Figure 8.1) are in valleys with steep side slopes producing high rates of runoff. The area contributing runoff to the West Branch, 8.3 sq mi, is relatively undeveloped, whereas the area contributing runoff to the East Branch, 7.5 sq mi, is highly developed. Channel slopes of the East Branch are also significantly flatter than those of the West Branch. The contrasting physical differences in slope and urbanization for the separate watersheds have counteracting effects on



8.2. Annual Flood Peak Frequency Data and HEC-1 Results



8.3. Recommended Discharge-Frequency Curve

travel time. As a result, flood peaks nearly coincide at the confluence of the two branches.

8.3 Frequency Analysis

Data and procedures contained in Guidelines for Determining Flood Flow Frequency, (Water Resources Council, 1976) were used to compute flood flow frequency curves at the USGS recording streamflow gages: West Branch Rahway River at Millburn, Rahway River near Springfield, and Rahway River at Rahway. The Pearson Type III distribution with log transformation of the flood data was used as the basic distribution for defining the annual flood series. Plotting positions were computed according to Weibull's formula, and an expected probability adjustment was applied to each computed frequency curve.

8.4 Unit Hydrograph Analysis

A unit hydrograph study was made for the gaged basins (Rahway River near Springfield and Rahway River at Rahway) to derive t_c and R, the Clark unit hydrograph parameters. The values obtained were used to develop a regional relationship for t_c and R based on the physiographic characteristics of the basins.

The optimization feature of HEC-1 was used to compute the best-fit loss rate and unit hydrograph parameters. Program input data for each storm event consisted of: (1) mean storm rainfall for the gaged basin under investigation, (2) hourly precipitation data from the recording stations which provided a distribution pattern for the mean rainfall, (3) basin drainage area, (4) percent of impervious area, and (5) the observed runoff hydrograph.

Multiple runs were made according to the procedures outlined in Addendum 1 of the HEC-1 manual, until the observed hydrographs were approximated by the computed hydrographs in shape, timing, and volume. The exponential loss rate function was used for these runs, and regional values of ERAIN and RTIOL (see Chapter 5 for definitions) were based on the optimization results of this study and on optimization runs for hydrologically similar basins in the vicinity of the study area.

The derived 1-hour unit hydrographs and unit hydrograph parameters are shown in Figures 8.4 and 8.5. The adopted values of t_c and R for the two gages, Rahway River near Springfield and Rahway River at Rahway, were taken as the average of the derived parameters. The significant difference in timing and peak flow rate between the optimized 1-hour unit hydrographs of Figure 8.4 is probably due to (1) movement of the storm of August 1971 up the basin such that the time of concentration was greater, and (2) channelization, specifically in the East Branch, which tends to increase the peak discharge rate and reduce the time of concentration.

The adopted unit hydrograph parameters are tabulated in Table 8.1 with data from other gaged basins considered hydrologically similar to the study area. The variables of Table 8.1 are defined in Chapter 6. The values of I and S were taken from reports by the U. S. Geological Survey (1974) and the U. S. Army Corps of Engineers (1976).

A multiple regression routine (HEC, 1970) was used to correlate t_c and R with various physiographic characteristics of a drainage basin. Imperviousness was included as one of the parameters because of the high degree of development in parts of the basin and because of the significant difference in development in the areas contributing runoff to the East and West Branches of the Rahway River. In the regression analysis, imperviousness was shown to be a significant physical parameter. Results are given in Table 8.2 together with each of the equations considered.

Equation 8.3 was adopted as the best relationship for determining t_c as a function of the other parameters, based on the partial determination coefficients for the individual parameters and a comparison of the standard errors of estimate for the various expressions evaluated in the analysis. The Clark parameter t_c for ungaged basins is therefore computed as

$$t_c = 8.29\ (1.0 + 0.03I)^{-1.28}\ (DA/S)^{0.28} \quad \ldots\ldots\ldots\ldots\ldots \quad (8.3)$$

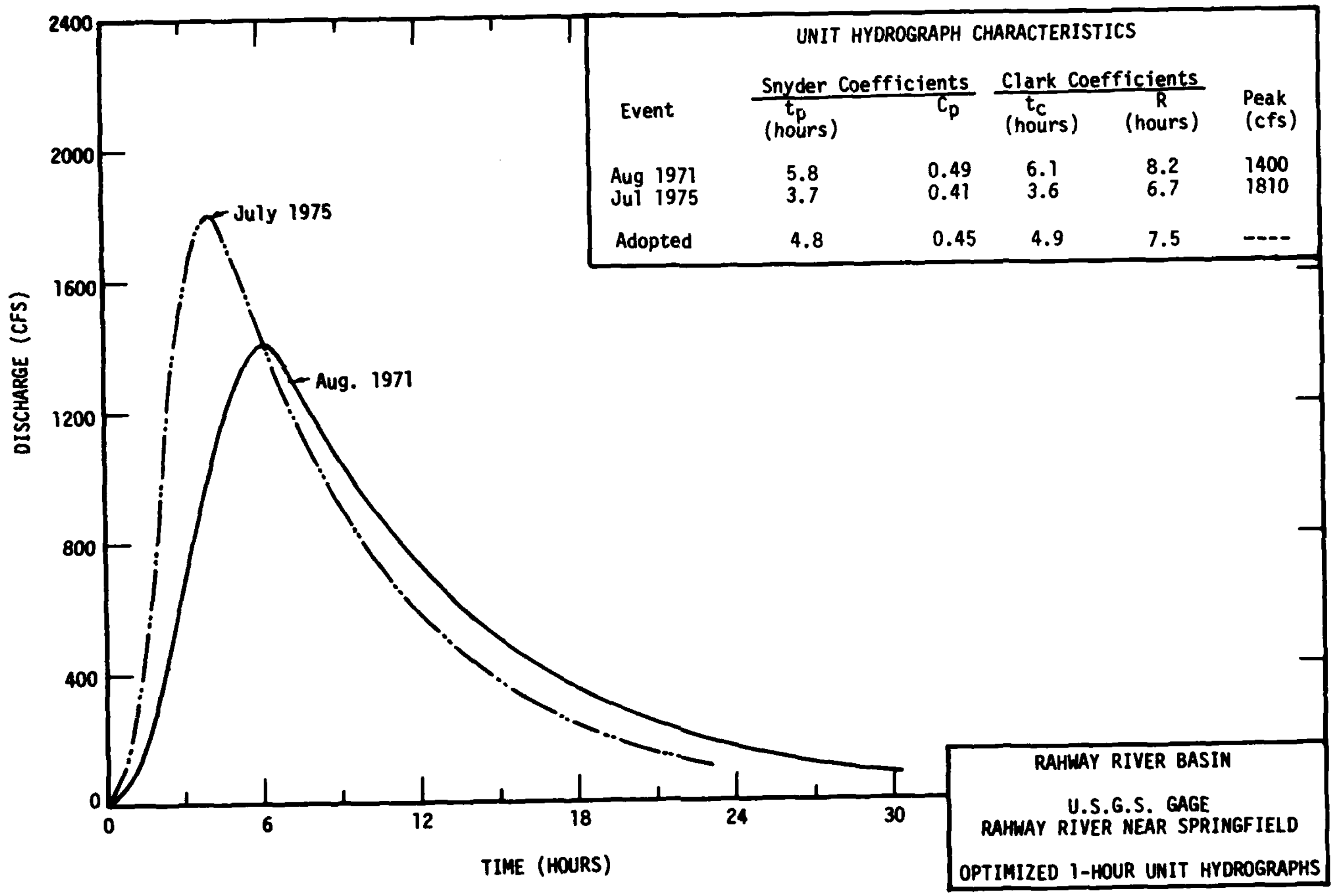
where t_c, I, DA, and S are as defined above. The transformation K = 1.0 + 0.03I was necessary to keep t_c from approaching infinity as I approached zero. It should also be noted that the slope characteristic, S, used for a given ungaged basin is the slope of that particular watercourse that yields

TABLE 8.1

REGIONAL ANALYSIS

SELECTED DATA USED IN THE MULTIPLE REGRESSION ANALYSIS RELATING t_c and R TO VARIOUS PHYSIOGRAPHIC CHARACTERISTICS OF THE RAHWAY RIVER

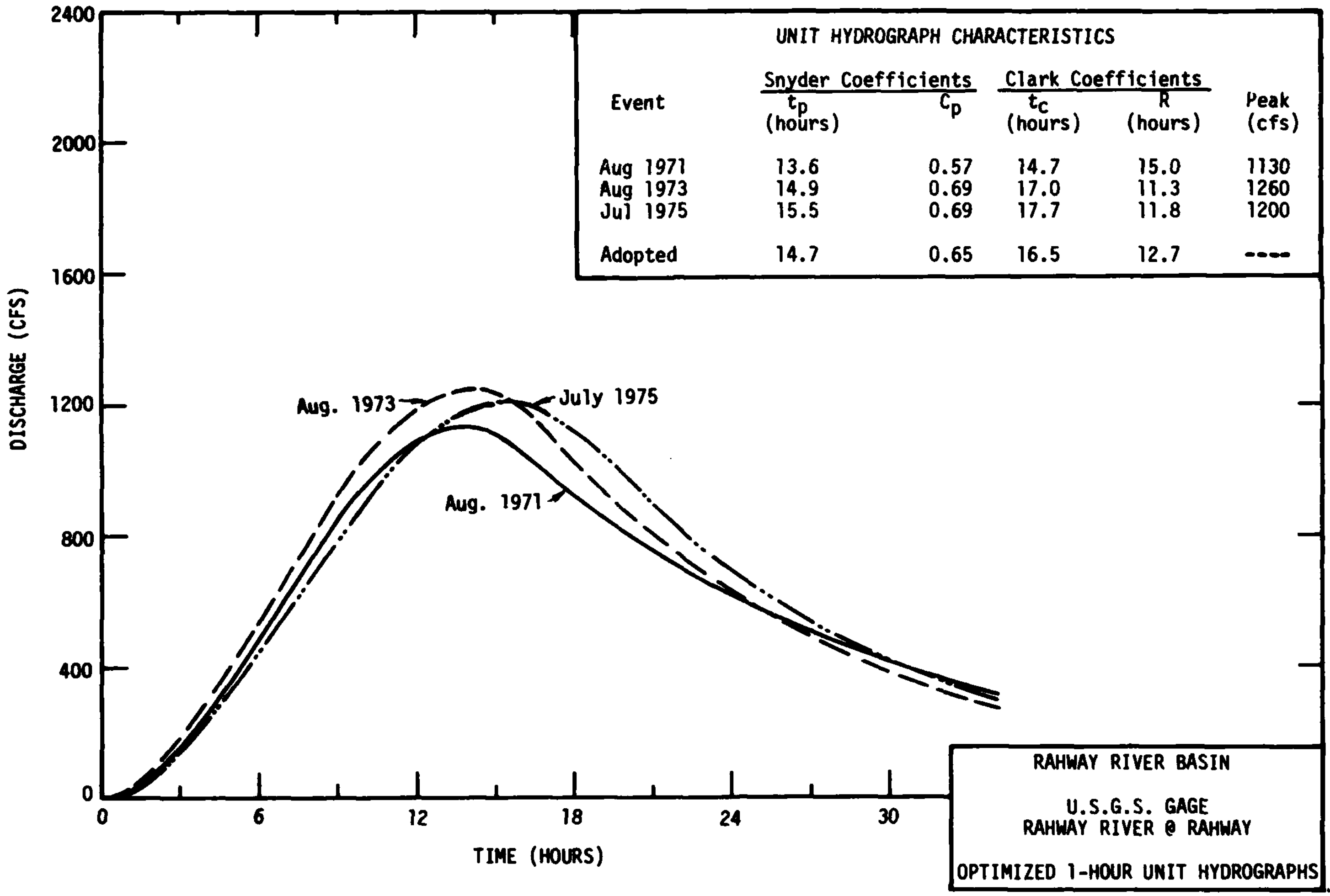
Gage Location	D A (sq mi)	S (ft/mi)	I (%)	t_c+R (hours)	t_c (hours)
1 Rahway River near Springfield, N.J.	25.5	14.6	26.0	12.4	4.9
2 Rahway River at Rahway, N.J.	40.9	8.8	24.0	29.2	16.5
3 Chester Creek near Chester, Pa.	61.1	22.6	9.5	11.9	8.1
4 Green Brook at Plainfield, N.J.	9.75	49.2	25.0	4.7	2.6
5 Robinsons Branch, Rahway R. at Rahway, N. J.	21.6	13.3	19.0	9.9	5.0
6 Elizabeth River at Elizabeth, N. J.	18.0	19.5	45.0	5.4	2.7



UNIT HYDROGRAPH CHARACTERISTICS

Event	Snyder Coefficients		Clark Coefficients		Peak (cfs)
	t_p (hours)	C_p	t_c (hours)	R (hours)	
Aug 1971	5.8	0.49	6.1	8.2	1400
Jul 1975	3.7	0.41	3.6	6.7	1810
Adopted	4.8	0.45	4.9	7.5	----

8.4. Optimized 1-Hour Unit Hydrograph for Rahway River Near Springfield



UNIT HYDROGRAPH CHARACTERISTICS

Event	Snyder Coefficients t_p (hours)	C_p	Clark Coefficients t_c (hours)	R (hours)	Peak (cfs)
Aug 1971	13.6	0.57	14.7	15.0	1130
Aug 1973	14.9	0.69	17.0	11.3	1260
Jul 1975	15.5	0.69	17.7	11.8	1200
Adopted	14.7	0.65	16.5	12.7	----

8.5. Optimized 1-Hour Unit Hydrograph for Rahway River at Rahway

the longest time of travel. It is not necessarily the slope of the longest watercourse.

The relationships of Table 8.2 containing R did not give reasonable results; therefore, the ratio

$$R/(t_c + R) = 0.65 \qquad (8.9)$$

was adopted. This ratio was used in the preliminary optimization work for the gaged basin (Rahway River near Springfield) before specific values of t_c and R were selected. Equation 8.9 reduces to

$$R = 1.85\, t_c \qquad (8.10)$$

TABLE 8.2

RESULTS OF MULTIPLE REGRESSION ANALYSIS ON A REGIONAL BASIS

Equation	Equation No.	Standard Error of Estimate	Correlation Coefficient $\bar{R}$	Coefficient of Determination $\bar{R}^2$
$t_c = 26.19\ I^{-0.53}\ S^{-0.29}\ (DA)^{0.23}$	(8.1)	0.0495	0.9710	0.9428
$t_c = 19.84\ I^{-0.50}\ (\frac{DA}{S})^{0.26}$	(8.2)	0.0358	0.9849	0.9701
* $t_c = 8.29\ K^{-1.28}\ (\frac{DA}{S})^{0.28}$	(8.3)	0.0269	0.9915	0.9831
$t_c = 4.14\ (\frac{DA}{S})^{0.39}$	(8.4)	0.1296	0.7800	0.6084
$(t_c + R) = 122.64\ I^{-0.42}\ S^{-0.55}\ (DA)^{0.09}$	(8.5)	0.1442	0.6844	0.4684
$(t_c + R) = 15.69\ I^{-0.21}\ (\frac{DA}{S})^{0.34}$	(8.6)	0.1161	0.8094	0.6552
* $(t_c + R) = 11.52\ K^{-0.67}\ (\frac{DA}{S})^{0.33}$	(8.7)	0.1054	0.8461	0.7159
$(t_c + R) = 7.98\ \frac{DA}{S}^{0.39}$	(8.8)	0.1093	0.8333	0.6944

* K = 1.0 + 0.03I

Optimization results for the USGS gage, Rahway River at Rahway, were not used in the preliminary determination of R or t_c for the ungaged basins because the channel storage in this basin is generally not representative of the ungaged areas.

8.5 Hypothetical Storms

Storms of various frequencies were developed for the area under investigation. The storms selected were those with exceedance intervals of 500, 100, 50, 25, 10 and 2 years, plus the Standard Project Storm (SPS).

The 500-, 100-, 50-, 25-, 10-, and 2-year storms were developed by procedures described in the Appendix. Average point rainfall depths were taken from the isopluvial maps for the study area location. The depths are tabulated in Table 8.3 for each storm for durations from 1 to 24 hours. The point rainfall depths were converted to 40-square-mile rainfall depths using the area-depth curves of the Appendix. A rainfall distribution similar to

TABLE 8.3

POINT RAINFALL FOR HYPOTHETICAL STORM EVENTS[1]

Duration (hours)	2-Year	10-Year	25-Year	50-Year	100-Year	500-Year[2]
	(inches)					
1	1.45	2.15	2.45	2.75	3.10	3.75
2	1.75	2.70	3.15	3.40	3.80	4.75
3	2.00	2.90	3.35	3.80	4.30	5.30
6	2.40	3.60	4.10	4.60	5.20	6.45
12	2.85	4.30	5.10	5.50	6.30	7.85
24	3.35	5.10	5.85	6.50	7.35	9.10

[1]Data taken from isopluvial maps contained in Technical Paper No. 40, Rainfall Frequency Atlas of the United States, U. S. Dept. of Commerce, Washington, D. C., 1961.

[2]Determined by extrapolation according to procedures contained in Technical Paper No. 40 for return periods longer than 100 years.

that for the SPS was used where the hour of largest precipitation is preceded by the second largest and followed by the third largest (see Table 8.4).

The rainfall-depth versus area data developed for the stream system procedure of HEC-1 are given in Table 8.5. This procedure automatically accounts for decreasing amounts of basin-average precipitation with increased basin size and is discussed in Addendum 2 of the HEC-1 Users Manual (HEC, 1973).

The Standard Project Storm rainfall for the study area was developed with procedures from EM 1110-2-1411 (U.S. Army Corps of Engineers, 1965). The 200-square mile, 24-hour precipitation index for the basin is 10.4 inches. Using a transposition coefficient of 1.0, the SPS rainfall distribution was developed for a tabulation interval of 1 hour. The 24-hour depth-area rainfall data were determined by using the 24-hour curve of Plate 9 of EM 1110-1-1411.

TABLE 8.4

POINT RAINFALL DISTRIBUTIONS FOR HYPOTHETICAL STORM EVENTS

Hours	2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
	(inches)					
1	0.02	0.04	0.04	0.05	0.05	0.06
2	0.02	0.04	0.04	0.05	0.05	0.06
3	0.03	0.05	0.04	0.06	0.06	0.06
4	0.04	0.06	0.05	0.07	0.06	0.07
5	0.04	0.06	0.06	0.08	0.09	0.10
6	0.05	0.07	0.07	0.09	0.11	0.1[illegible]
7	0.06	0.11	0.15	0.13	0.16	0.21
8	0.07	0.11	0.15	0.14	0.17	0.22
9	0.07	0.11	0.16	0.14	0.18	0.23
10	0.08	0.12	0.17	0.15	0.18	0.23
11	0.08	0.12	0.18	0.16	0.19	0.24
12	0.09	0.13	0.19	0.18	0.22	0.27
13	0.12	0.20	0.20	0.24	0.27	0.35
14	0.14	0.23	0.25	0.29	0.32	0.42
15	0.30	0.55	0.70	0.65	0.70	1.00
16	1.45	2.15	2.45	2.75	3.10	3.75
17	0.25	0.25	0.27	0.40	0.50	0.55
18	0.14	0.22	0.23	0.27	0.31	0.38
19	0.07	0.10	0.11	0.13	0.14	0.18
20	0.06	0.09	0.09	0.12	0.13	0.15
21	0.05	0.08	0.07	0.11	0.12	0.14
22	0.04	0.08	0.07	0.09	0.09	0.11
23	0.04	0.07	0.06	0.08	0.08	0.09
24	0.04	0.06	0.05	0.07	0.07	0.08
TOTAL	$\boxed{3.35}$	$\boxed{5.10}$	$\boxed{5.85}$	$\boxed{6.50}$	$\boxed{7.35}$	$\boxed{9.10}$

TABLE 8.5

24-HOUR DEPTH-AREA RAINFALL DATA
FOR HYPOTHETICAL STORMS

Drainage Area (sq mi)	Rainfall in inches					
	2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
0[1]	3.35	5.10	5.85	6.50	7.35	9.10
5	3.33	5.07	5.82	6.47	7.31	9.05
10	3.32	5.05	5.79	6.44	7.28	9.01
20	3.28	4.99	5.72	6.36	7.19	8.90
30	3.24	4.93	5.66	6.29	7.11	8.80
40	3.21	4.88	5.60	6.22	7.03	8.71
50	3.18	4.85	5.56	6.18	6.98	8.65

[1] Point rainfall for hypothetical storm events.

8.6 Basin Hydrologic Model for Hypothetical Storms

The starting discharge (STRTQ) adopted for each subbasin was 1.00 cfs/sq mi. Recession discharges began at a value of 5 percent of the peak discharge(QRCSN) with RTIOR set equal to 2.00. These parameters are averages of the STRTQ-, QRCSN- and RTIOR-values used in the calibration analysis. Regional values of ERAIN and RTIOL of 0.70 and 2.00, respectively, were used in the precipitation loss rate function. Values of STRKR and DLTKR were initially estimated for each hypothetical storm event.

The basin model was run using the stream system procedure of HEC-1 (described in Addendum 2 of the HEC-1 Users Manual) for each hypothetical storm event. The stream system computation procedure greatly simplifies the computation of multiple flood events in a basin through the use of "index hydrographs." The peak flows as computed at each USGS streamflow gage location were compared with those obtained from the discharge-frequency curves obtained by statistical analysis. The values of the loss rates (STRKR and DLTKR) were adjusted until the computed peak flows approximated the respective flows obtained in the frequency analysis.

In the stream-system subroutine of HEC-1, only one rainfall distribution at a time can be entered as input. Since the distribution patterns for the 500-, 100-, 50-, 25-, 10-, and 2-year hypothetical storms are functions of drainage-area size, distributions were determined for point rainfall and 40-square-mile rainfall. HEC-1 runs were made with each distribution for a particular hypothetical storm. A logarithmic interpolation was then made between the two results to determine the discharge for a specified drainage area size.

8.7 Results of the Hydrologic Analysis

The recommended peak discharges at various stream locations are tabulated in Table 8.6. If discharges are needed for exceedance intervals which are not given on the table, they can be obtained by plotting the discharges at a particular location on log-probability paper as in Figure 8.3.

TABLE 8.6

RECOMMENDED PEAK DISCHARGES AT VARIOUS STREAM LOCATIONS FOR HYPOTHETICAL FLOODS

Description of Stream Location	Drainage Area (sq mi)	SPF	500-Year	100-Year	50-Year	10-Year	2-Year
		(cfs)					
West Branch Rahway River at Diamond Mill Pond (subareas 1 and 2)	7.13	5,500	4,150	2,650	2,050	1,100	600
Immediately upstream of confluence (subareas 1, 2 and 3)	8.28	5,800	4,300	2,800	2,100	1,150	650
Rahway River immediately downstream of confluence	15.81	11,500	8,600	4,950	3,900	2,200	1,050
Immediately upstream of confluence (subareas 1 thru 7)	17.85	11,100	8,350	4,800	3,700	2,000	1,000
Rahway River immediately downstream of confluence with Van Winkle Brook (subareas 1 thru 8)	23.22	14,500	10,900	6,050	4,050	2,400	1,800
Rahway River at Springfield gage (subareas 1 thru 9)	25.49	14,900	11,200	6,050	4,600	2,400	1,300
Immediately upstream of confluence with Nomahegan Brook (subareas 1 thru 10)	26.51	12,850	9,700	5,000	3,750	2,000	1,000
[illegible] immediately downstream of confluence with Nomahegan Brook ([illegible] 1 thru 11)	31.00	15,100	11,500	6,100	4,700	2,700	1,400
[illegible] Railroad [illegible] thru 12)	36.49	14,800	11,400	6,300	4,850	2,700	1,450
[illegible] thru 13)	40.93	14,900	11,700	6,600	5,100	2,700	1,450

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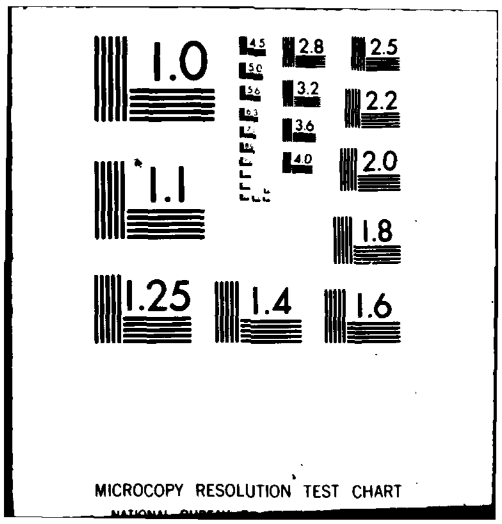
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MICROCOPY RESOLUTION TEST CHART

CHAPTER 9

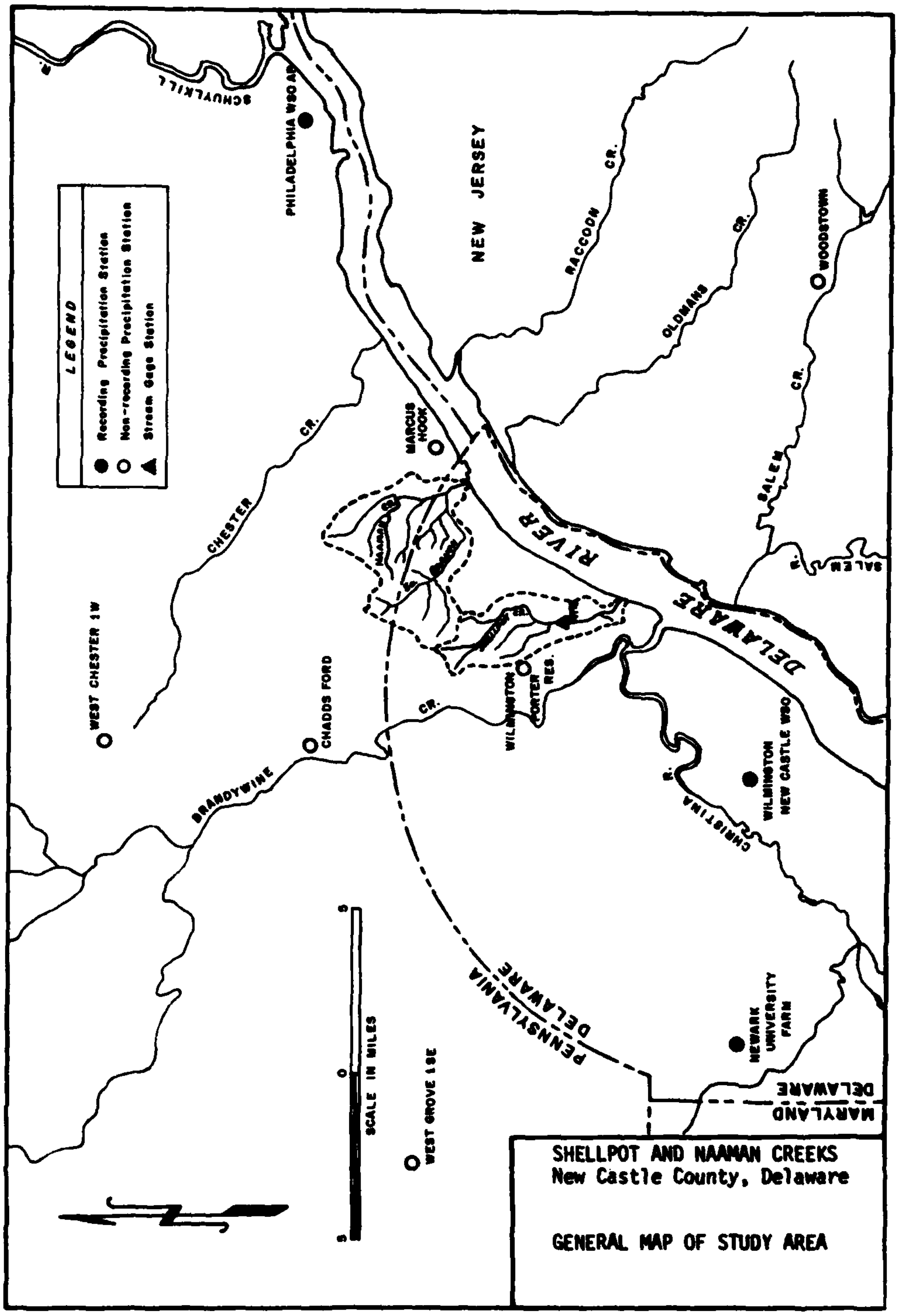
STUDY II - ANALYSIS WITH SOME STREAMFLOW DATA AVAILABLE

Some data (such as flow data from a single stream gage) are available for the watershed in many cases. Usually, however, the gage is not at the location at which the discharge-frequency relationship is needed. An example of such a situation is Shellpot Creek, a 9.43-sq mi basin near Wilmington, Delaware. The study on which this example is based is described in a Special Projects Memorandum by the Hydrologic Engineering Center (1976b). The hydrological data were developed for use in Special Flood Hazard Information reports. Basic rainfall and runoff data were analyzed, and the HEC-1 model was calibrated on several observed events and applied to specific hypothetical events. The following paragraphs describe the procedures and assumptions which were used to develop recommended discharges at selected exceedance frequencies for this basin.

9.1 Basin Description

Shellpot Creek is located in the northeastern corner of the state of Delaware. Figure 9.1 is a general map of the study area. Shellpot Creek enters the Delaware River at Wilmington, Delaware. It drains an area of 9.43 square miles, all within New Castle County, Delaware. The basin is nearly completely urbanized, and the predominant type of land use is single-family residential, with industrial and commercial development concentrated primarily in the lower portions of the basin near the Delaware River.

Channel slopes vary from less than 25 ft per mi in the lower reaches of the stream to over 200 ft per mi in the steeper slopes near the watershed boundary. The main channel slope of Shellpot Creek (between points 10 and 85 percent of the distance along the longest watercourse) is 40 ft per mi. Land surface slopes average about 3 to 5 percent, and soils are predominantly of the moderate-infiltration soils group. Mean annual precipitation is about 43 in, and average annual runoff for Shellpot Creek during the 29-year period 1946-74 was 17.5 in. Streamflow data are available at one location on Shellpot Creek (at Wilmington, Delaware). The drainage area at this point is 7.46 sq mi. Continuous records are available from December 1945 to the present.



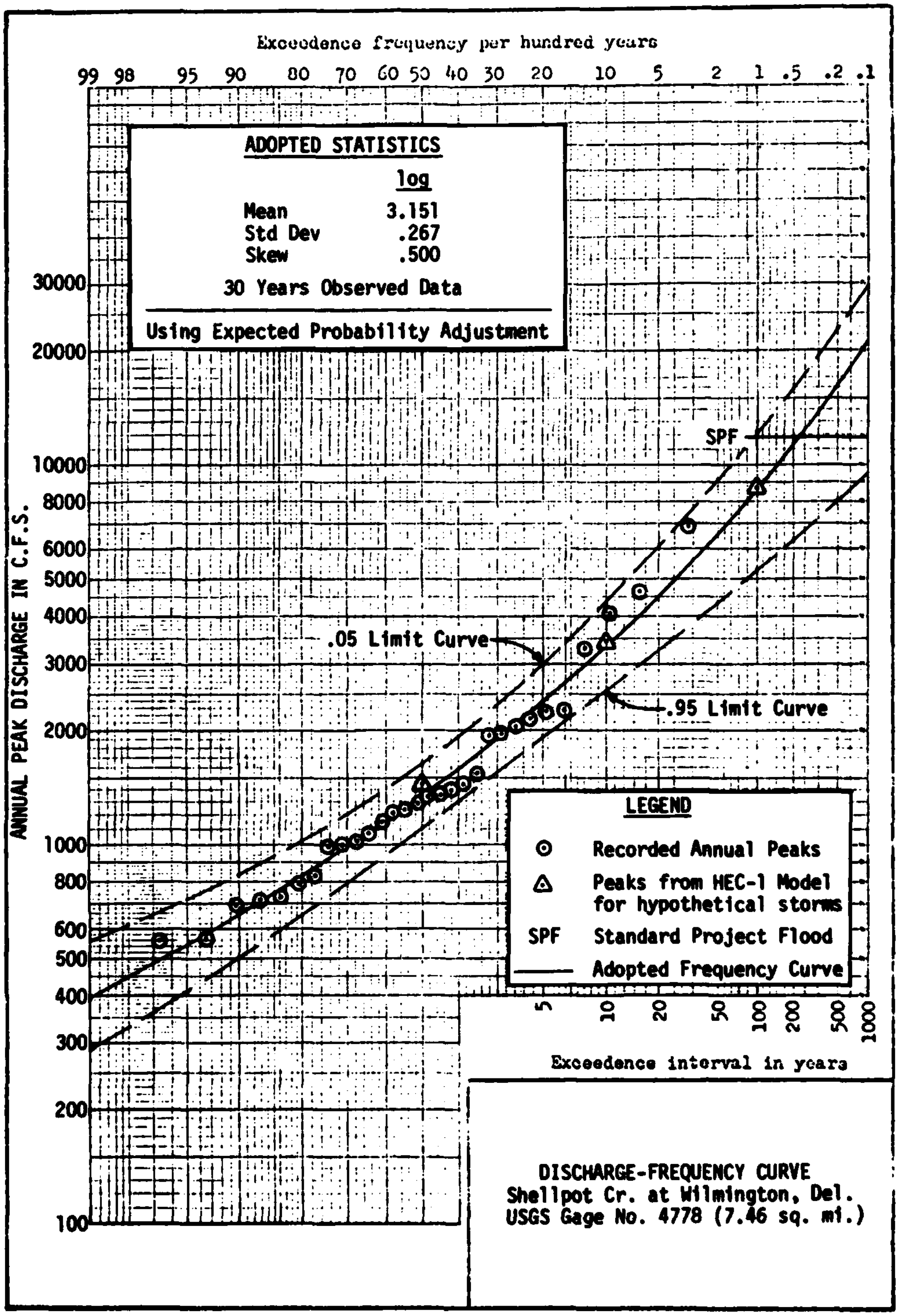
9.1. Shellpot Creek Study Area

9.2 Observed Frequency (Observed Data)

A discharge-frequency analysis was made on the 30-year period 1946-75 based on annual maxima. Water Resources Council Guidelines procedures (WRC, 1976) were applied using a weighted skew value of 0.5. Figure 9.2 shows a plot of these data based on the Weibull plotting position equation. The computed mean, based on the log transformation of the data, is 3.151 (or 1,400 cfs), and the standard deviation is 0.267. Nearby basins having streamflow records with longer periods of record are almost nonexistent. Chester Creek, about 10 miles northeast, has continuous records from 1932, but has a much larger drainage area (61.1 sq mi) and very little urbanization. Correlation of annual peak discharges for Shellpot and Chester Creeks resulted in a correlation coefficient of 0.428. If the Water Resources Council Guidelines are followed, the correlation coefficient would need to be equal to or greater than 0.93 in order to justify adjustment of the gaged annual-series discharge statistics of Shellpot Creek.

The only other nearby station with a longer term of record is Brandywine Creek at Chadds Ford, Pennsylvania, with records from 1911 to 1953 and from 1962 to the present. The area associated with this station is 287 square miles, and there is essentially no urbanization. Because of the great difference in basin size, no correlation studies were made. The curve in Figure 9.2 is considered to be the best estimate of the discharge-frequency characteristics for Shellpot Creek and the best guide for developing reasonable estimates at other locations within the basin.

The six highest recorded events at the Shellpot Creek gage have peak discharges that are all greater than the 5-year event (20 percent chance of annual exceedance). Data for these events are given in Table 9.1. Because of the small size and nearly complete urbanization of this basin, it is reasonable to expect less variability in initial rainfall loss by infiltration at various times of the year; therefore, no additional search of rainfall records was made to find storms that were more severe than those resulting in these peak discharges.



9.2. Discharge-Frequency Curve for Shellpot Creek

TABLE 9.1

DATA FOR HISTORICAL EVENTS

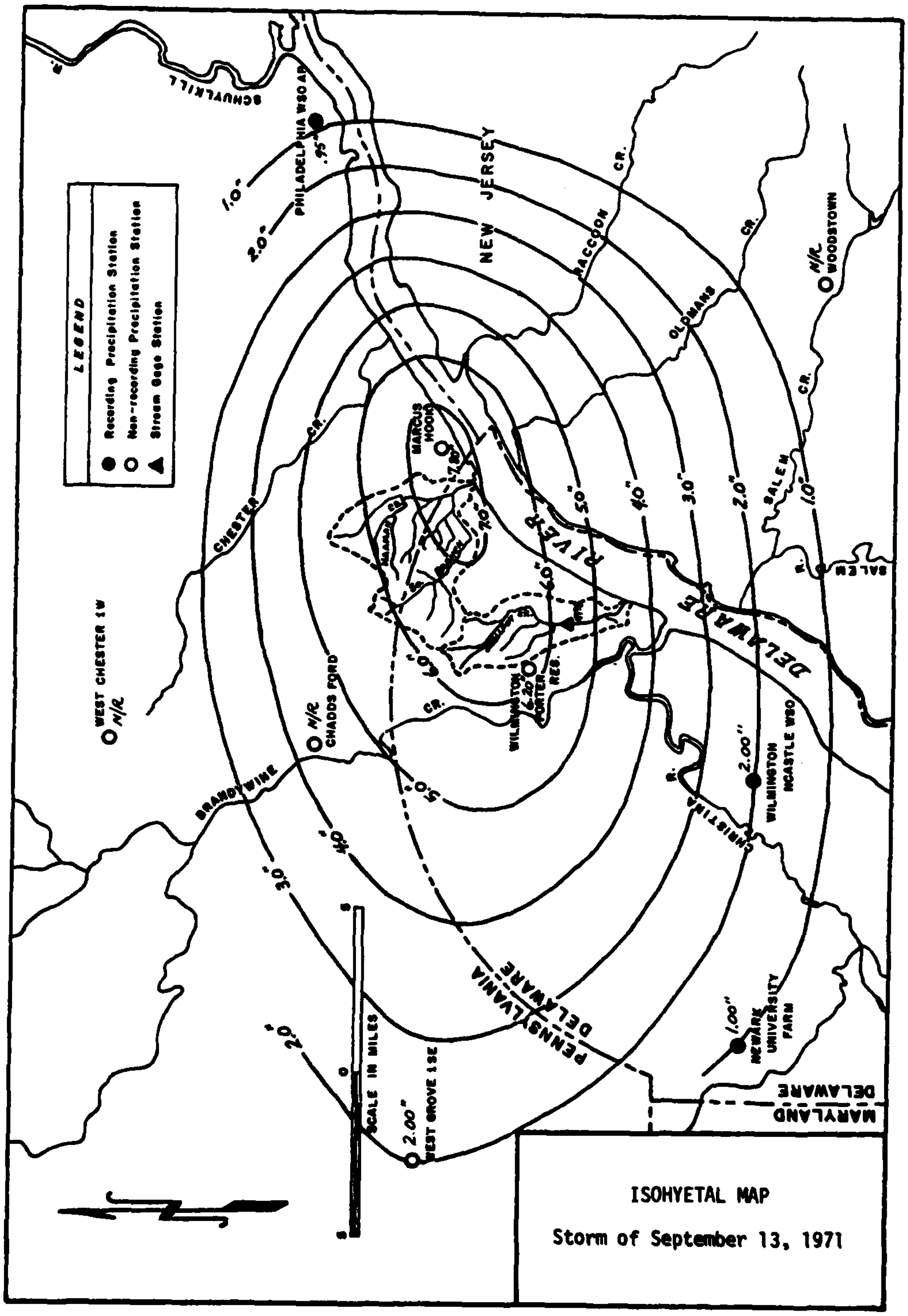
Date of Peak	Peak (cfs)	AVERAGE FLOW 1 Day (cfs)	3 Day (cfs)	TP[1/] (hrs)	RR[2/] (ft/hr)
13 Sep 1971	6850	1300	536	2	4.5
27 Aug 1967	4650	577	263	2	5.4
9 Jul 1952	4080	643	225	3.5	2.5
23 Aug 1974	3300	386	136	4.5	2.0
18 Aug 1955	2280	279	110	2	3.7
22 Jun 1972	2240	501	183	3	2.8

1/ Time from start of rise to peak discharge
2/ Maximum rate of rise

9.3 Unit Hydrograph Studies

Five of the six events listed above were analyzed in detail with HEC-1. Isohyetal maps were prepared for each storm analyzed. Recording and nonrecording gage locations are shown on a typical isohyetal map in Figure 9.3. A great deal of judgment was required in developing isohyetal patterns for each storm because there is a substantial variation in the total storm rainfall amounts observed at the various gages surrounding the basin. The distribution of storm rainfall in time is subject to considerable error when based on stations several basin widths away. Total basin-average precipitation was estimated for each storm event. The HEC-1 unit hydrograph and loss-rate optimization option was used to determine 15-minute unit hydrograph and loss rate parameters based on observed discharge hydrographs. Rainfall and runoff data for the five storms are presented in Table 9.2.

Unit hydrograph data and basin characteristics are presented in Table 9.3. For the full basin at the gage the average Snyder t_p is 2.1 hours, and C_p is 0.77. Figure 9.4 represents a typical reconstituted



9.3. Typical Isohyetal Map

TABLE 9.2

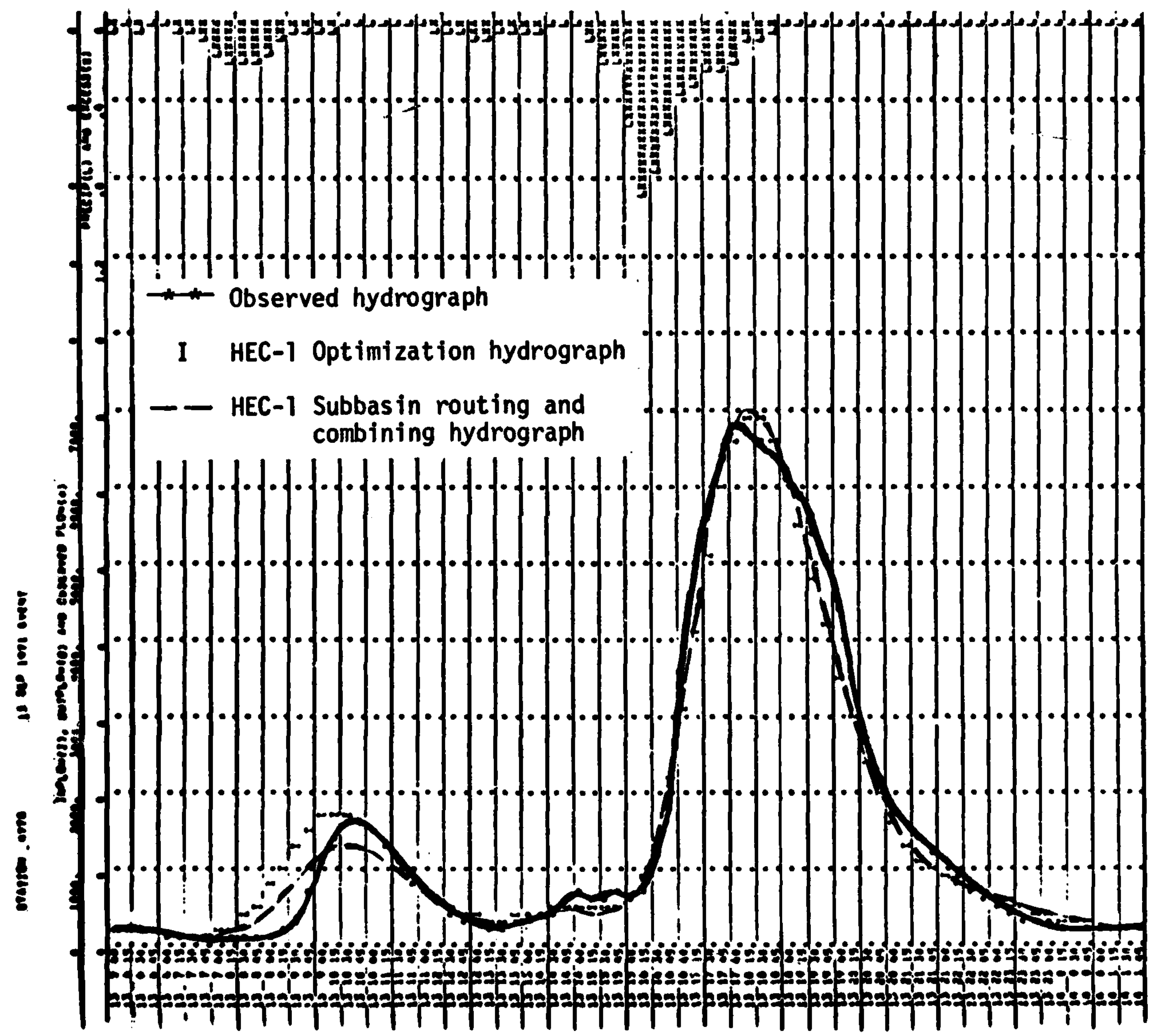
RAINFALL AND RUNOFF FOR STORM EVENTS

Event	Basin Average Rainfall and Runoff Shellpot Cr. at Wilmington, Del. (7.46 sq mi) Rainfall (inches)	Runoff (inches)
18-19 August 1955	2.44	1.52
27 August 1967	2.24	2.07
13 September 1971	6.36	6.24
22 June 1972	3.74	2.43
22-23 August 1974	3.85	1.87

TABLE 9.3

SHELLPOT CREEK BASIN
PHYSICAL CHARACTERISTICS AND
UNIT HYDROGRAPH DATA FOR SUBBASINS

Subbasin Number	DA (sq mi)	L (miles)	L_{ca} (miles)	Slope 10-85 (ft/mi)	Percent Impervious I	t_p (hours)
1	0.64	1.00	0.40	73	23	0.66
2	0.71	1.70	0.80	48	30	0.96
3	0.66	1.80	1.00	61	42	1.04
4	0.94	2.20	1.10	50	35	1.13
5	0.75	1.70	1.00	86	30	1.02
6	1.11	2.10	1.10	97	20	1.12
7	0.93	2.35	1.30	119	25	1.22
8	1.34	3.25	1.60	111	30	1.43
9	0.38	1.05	0.52	240	30	0.73
Gage	7.46	6.10	3.10	40	30	2.10
10	0.41	1.25	0.50	186	30	0.75
11	1.56	2.40	1.40	25	35	1.26



13 SEPTEMBER 1971
SHELLPOT CR. AT WILMINGTON, DEL.

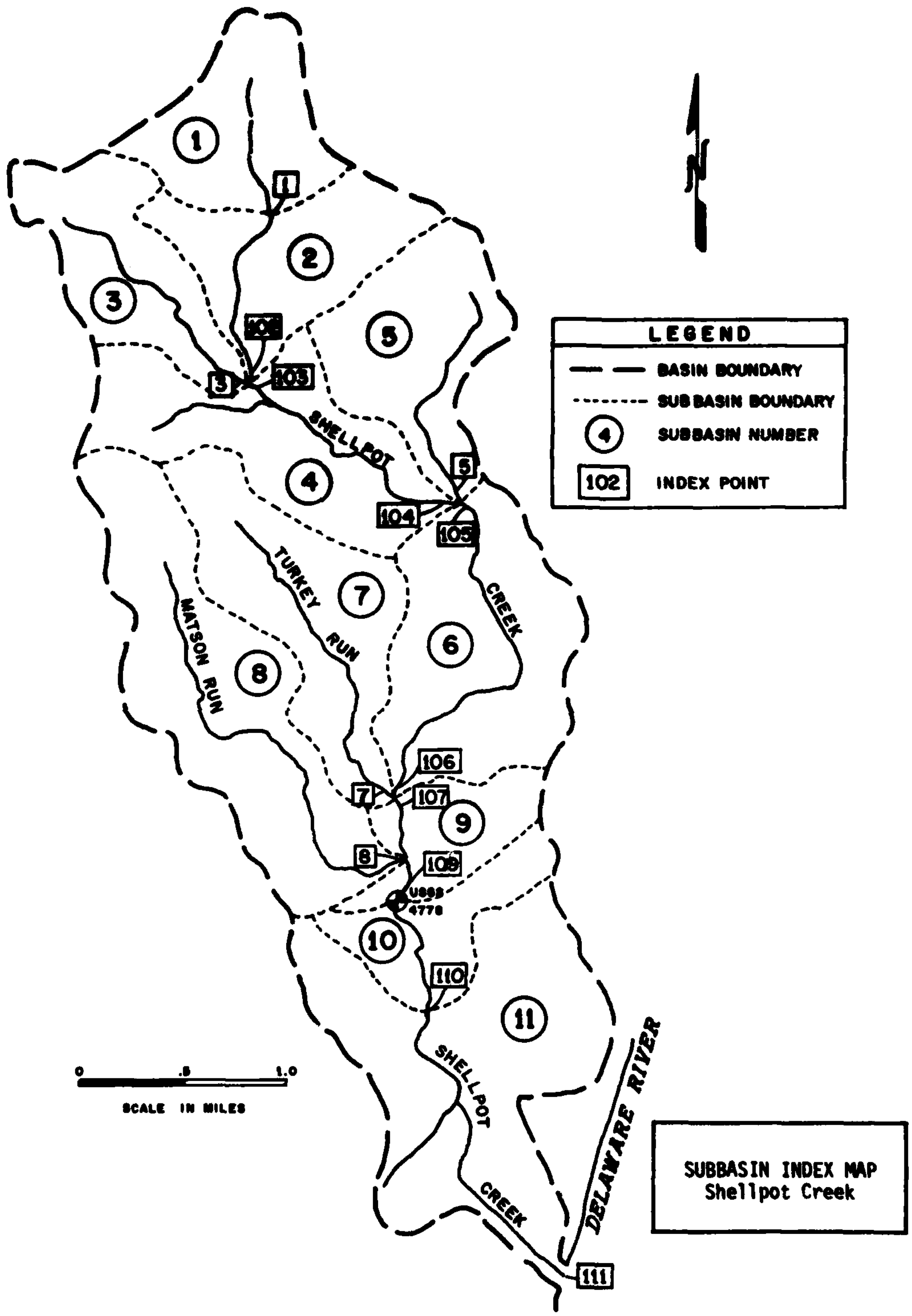
9.4. Simulated and Observed Hydrographs

hydrograph along with the observed hydrograph. Rainfall distribution was based on mass curve plots of available recorder data and an adjusted average curve. Fifteen-minute values were then interpolated from the curves. The rainfall exponent parameter ERAIN was set to zero and the loss rate parameter RTIOL was set equal to 1.0. This forced the program to optimize losses using essentially an initial loss plus a constant loss. The values of t_p and C_p were reasonably stable for the five events studied, but the loss parameters displayed more variability due to an inability to match antecedent soil moisture conditions and the sensitivity of the parameters to errors in estimating total rainfall and the rainfall distribution. The overall hydrograph reconstitutions are acceptable, and differences between observed and reconstituted hydrographs can be explained as random errors, resulting primarily from non-uniformity of rainfall distribution.

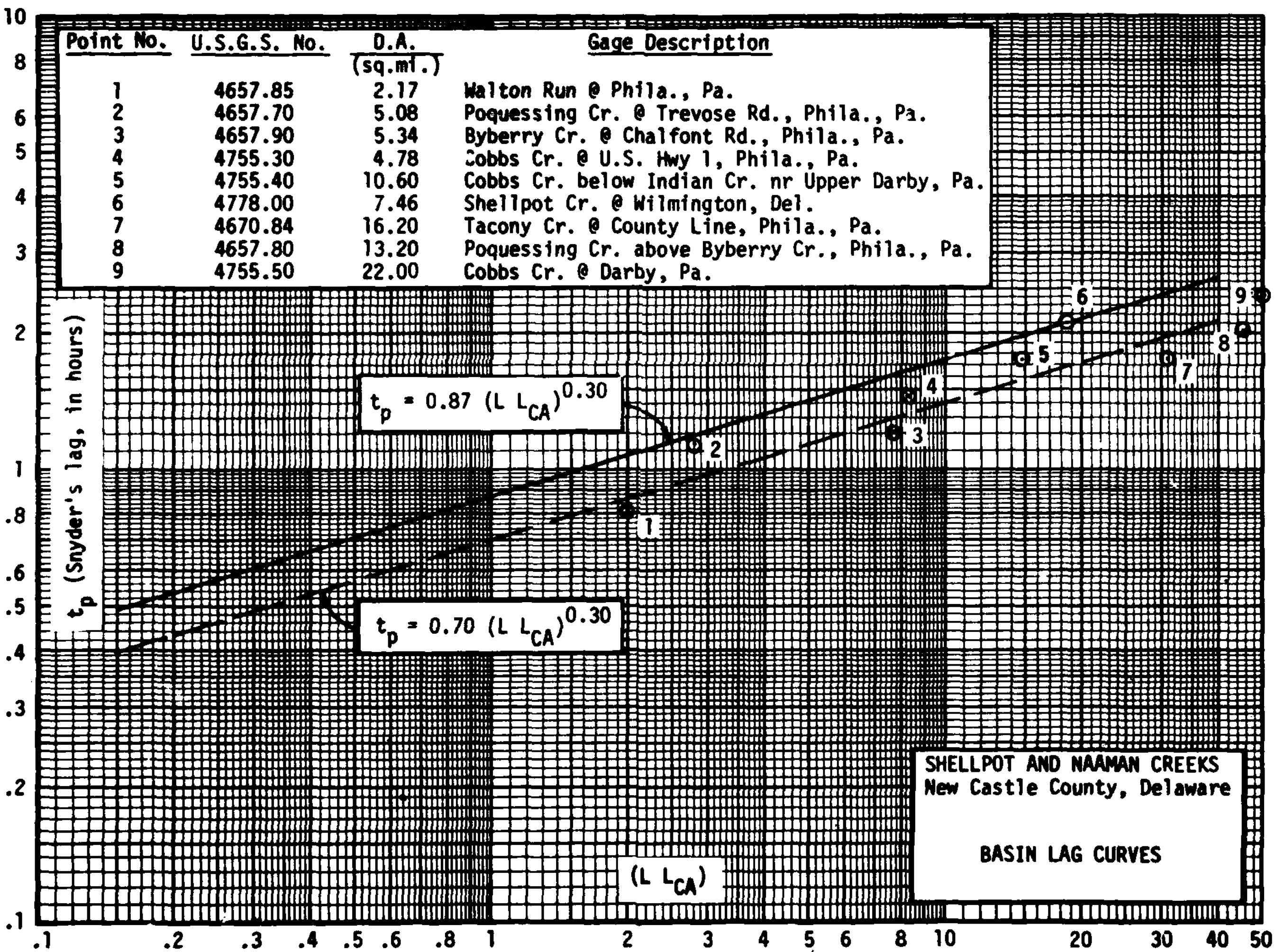
9.4 Basin Subdivision and Model Calibration

In order to determine probable flood discharges at other locations within Shellpot Creek, the basin was subdivided as shown in Figure 9.5. Index points were selected to coincide with major tributaries and spaced along the main watercourse in such a way as to result in only minor changes in discharges between mainstem index points. The percent imperviousness was estimated from areal photographs and published information.

To develop appropriate unit hydrographs for each sub-basin, a relationship between the basin physical characteristics of stream length (L) and length to center of basin (L_{ca}) versus unit hydrograph lag (t_p) was developed from a comparison of Shellpot Creek parameters with those for gaged urban basins in Philadelphia, Pennsylvania. Also given consideration was the t_p versus L L_{ca} relation adopted in the Red Lion Creek Study described in Chapter 10. Initial estimates for t_p were based on a line drawn through the Shellpot Creek five-storm average value which was drawn parallel to lines describing relations adopted in other Delaware and southeastern Pennsylvania stream studies. This line (Fig. 9.6) has the equation $t_p = 0.87\ (L\ L_{ca})^{0.30}$. A Snyder peaking parameter C_p of 0.77 was adopted for all subbasins. It is the average value determined in the optimization studies discussed above. Starting discharge (STRTQ) varied from 0.13 to 30 cfs per sq mi for the various events and was based on gage data. For the final loss-rate



9.5. Basin Subdivision



Point No.	U.S.G.S. No.	D.A. (sq.mi.)	Gage Description
1	4657.85	2.17	Walton Run @ Phila., Pa.
2	4657.70	5.08	Poquessing Cr. @ Trevose Rd., Phila., Pa.
3	4657.90	5.34	Byberry Cr. @ Chalfont Rd., Phila., Pa.
4	4755.30	4.78	Cobbs Cr. @ U.S. Hwy 1, Phila., Pa.
5	4755.40	10.60	Cobbs Cr. below Indian Cr. nr Upper Darby, Pa.
6	4778.00	7.46	Shellpot Cr. @ Wilmington, Del.
7	4670.84	16.20	Tacony Cr. @ County Line, Phila., Pa.
8	4657.80	13.20	Poquessing Cr. above Byberry Cr., Phila., Pa.
9	4755.50	22.00	Cobbs Cr. @ Darby, Pa.

9.6. Regional Parameters

ameters, the initial-loss volume ranged from 0.4 to 2.0 in. and constant s-rates from 0.06 to 0.65 in. per hr for the five storm events. A stant ratio of the peak (18 percent of the peak discharge) was selected the start of recession discharge (QRCSN), and a rate-of-recession ameter (RTIOR) equal to 3.0 was used for each subbasin based on a ninute time interval.

Channel Routing Criteria

Channel routing is required in the model to provide the correct nslation of the flood hydrograph along the stream from index point to ex point. Routing provides the timing and attenuation which reflect the rage characteristics of the channel and overbank sections of the stream ch. Since detailed stream cross-section and bridge data were unavailable, roximate routing methods were used. Channel reach lengths and slopes were imated from USGS 1:24,000 scale maps that had a contour interval of 10 A value of average velocity was then estimated for each channel reach ed on Manning's equation and assuming steady flow with an average slope al to the channel slope. The Muskingum routing method was used with a charge weighting coefficient of X = 0.3. A reach travel time K in hours estimated from the mean velocity. Adjustments were made to K as essary to make the routed and combined hydrographs for the nine ributing subbasins agree with the observed runoff events at the stream :.

Hypothetical Storm Runoff Estimates

Hypothetical storm data were developed by the procedures described in ter 4 and the Appendix of this report. Twenty-four-hour storms were loped for return periods of 2-years, 10-years, 100-years, and for the dard Project Storm (SPS). The computed 24-hour point rainfall values assumed to be applicable to areas of up to 1 sq mi. Adjustments were to all durations (15-minute to 24-hour) for areas of 5, 10, and 15 sq and the storms were distributed into successive 15-minute periods. These sted and distributed data were used in the stream-system option of the l computer program to generate consistent hydrographs for each subbasin combining point.

The model results at the gage were compared with the adopted discharge-frequency curve shown in Figure 9.2, and adjustments were made to loss rate parameters until reasonable agreement was obtained. However, even with zero loss, the 100-year and Standard Project Storms could not be made to give peak discharges as high as those given by the curve. It is generally accepted that unit hydrographs have a shorter t_p when intense storms occur over basins having confined flood plains. Therefore, it was rationalized that values of t_p for the 100-year and the Standard Project Flood (SPF) could reasonably be adjusted by a factor of 0.8. An adjustment factor of 0.8 was also applied to the channel-routing time-of-travel K values. This allowed reasonable loss-rate parameters to be adopted and resulted in values consistent with the statistical analysis of observed annual maximum discharges and SPF shown in Figure 9.2. Final loss parameters (STRTL and CNSTL) used for all subbasins were:

	2-Yr	10-Yr	100-Yr	SPF
Initial Loss (STRTL) in in.	2.0	1.5	0.5	0.5
Constant Rate (CNSTL) in in./hr	1.0	0.8	0.1	0.05

9.7 Regional Discharge-Frequency Comparisons

Regional discharge-frequency characteristics were determined using data from studies of other basins in the area (Hydrologic Engineering Center, 1974). The following correlation relationship for these regional data was developed:

$$\log Q_m = C_m + 0.87 \log (DA) \quad (9.1)$$

where

Q_m = geometric mean of the annual flood peaks in cfs,
C_m = "mapping coefficient" for a particular basin in the region, and
DA = drainage area in sq mi

A regression equation which contained the main channel slope in addition to DA produced a slightly better correlation (the regression coefficient R^2 was 0.897 when the slope was included, versus 0.871 for the relationship

expressed by equation 9.1) The equation with the slope was not used, however, because it was felt that the great deal of effort needed to determine S for all the ungaged streams was not justified for such a small increase in R^2.

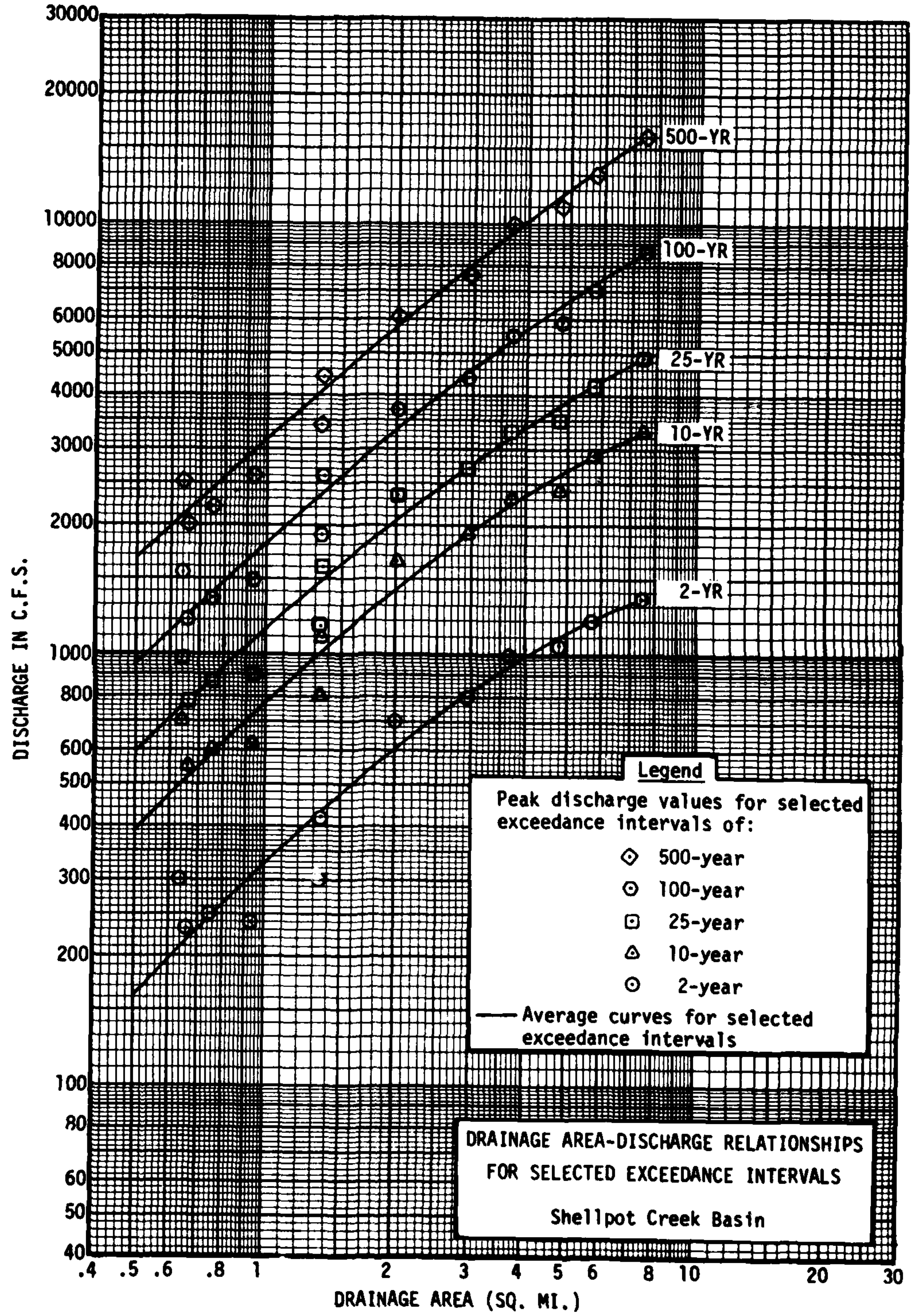
The mapping coefficient C_m is the difference between the observed and computed mean logarithm of the flow. For the Shellpot Creek data the coefficient was relatively high (C_m = 2.392). This was attributed to the high percentage of urbanized area within the basin in comparison to the other basins in the region.

The equation developed in the regional studies for estimating S the standard deviation of the logarithms of annual maximum discharges is:

$$S = C_s - 0.05 \log (DA) \quad \ldots\ldots\ldots\ldots \quad (9.2)$$

where C_s is a mapping coefficient for the standard deviation. The C_s value was determined to be 0.311. This value is consistent with data from adjacent basins.

Computed values of log Q_m and S for each index point were used as input to a regional discharge-frequency program, using an assumed equivalent length of record of 20 years and a regional skew of 0.5. Results were plotted and used to evaluate results of the hypothetical-storm approach discussed above. Since the regression equation does not account for differences in land use, channel slopes, or basin shape, it was felt that the answers determined from the HEC-1 model were better. Consequently, the flows computed from the HEC-1 model runs were given more weight in the preparation of recommended curves. Discharges based on the recommended curves are given in Table 9.4 for all index points. The relatively wide scatter displayed on plots of peak discharges versus drainage area in Figure 9.7 are attributable to the sensitivity of peak discharges to differences in land use, basin shape, and channel slopes.



9.7. Drainage Area - Discharge Relationships

TABLE 9.4

RECOMMENDED PEAK DISCHARGES AT SELECTED INDEX POINTS
(SHELLPOT CREEK BASIN)

Index Point	Location Description	D.A. (sq mi)	Peak Discharges (in C.F.S.) for Selected Exceedance Intervals 2-Year	10-Year	25-Year	100-Year	SPF	500-Year
1	Subbasin No. 1	0.64	300	700	980	1530	1700	2500
102	Subbasins 1 and 2	1.35	420	1100	1600	2600	3000	4400
3	Subbasin No. 3	0.66	230	550	780	1200	1400	2000
103	Subbasins 1 thru 3	2.01	700	1650	2350	3700	4500	6100
104	Subbasins 1 thru 4	2.95	800	1900	2700	4400	5500	7600
5	Subbasin No. 5	0.75	250	600	860	1350	1600	2200
105	Subbasins 1 thru 5	3.70	1000	2300	3300	5500	6900	10000
106	Subbasins 1 thru 6	4.81	1050	2400	3500	5900	7900	11000
7	Subbasin No. 7 (Turkey Run)	0.93	240	620	900	1500	1800	2600
107	Subbasins 1 thru 7	5.74	1200	2900	4200	7000	9500	13000
8	Subbasin No. 8 (Matson Run)	1.34	300	800	1170	1900	2500	3400
109	Subbasins 1 thru 9 (USGS Gage) (Shellpot Cr. at Wilmington, Del.)	7.46	1350	3300	4900	8600	12000	16000

1/Refer to Figure 9.5 for subbasin and index point locations.

9.8 Conclusions

Discharges for primary index points on Shellpot Creek are summarized in Table 9.4 for selected exceedance intervals. Discharges at other exceedance intervals can be estimated from plots of the data. If discharge estimates are needed for additional index points along the stream, discharges can be determined by interpolation based on respective drainage areas and the nearest upstream and downstream index point. The 100-year peak discharge estimate for this basin appears high (approximately 1,200 cfs per sq mi) when compared with other streams in Delaware and southeastern Pennsylvania, but with 30 years of observed data on Shellpot Creek it is reasonable to use the discharge-frequency curve from these data as the best indication of what could occur on the urbanized subbasins.

CHAPTER 10

CASE STUDY III - ANALYSIS WITH NO AVAILABLE DATA

The hydrologist is frequently faced with the situation in which either no gage is available in the basin studied or the record is so short that frequency relationships cannot be determined. Typifying such a situation is the example presented here, Red Lion Creek Basin (HEC, 1976c). The single gage available had a record of only nine years at the time of the study, and this record was from a crest stage gage with no capability to define the complete hydrograph.

Regional data were used to develop relationships for unit hydrograph parameters to determine discharge-frequency relationships for the basin for use in a Flood Plain Information Report. Loss rates adopted were based on results of the HEC-1 rainfall-runoff model calibration for one of the subbasins within the study area where the limited streamflow records were available.

10.1 Study Outline

The steps followed in the study were as follows:

(1) A study map was developed using USGS topographic maps (scale 1:24,000), and the basin boundary was defined. Index stations were located along the stream reaches where discharge-frequency data were required.

(2) The watershed was subdivided into subbasins at each of the index stations and/or required control points, and physiographic characteristics were determined for each subbasin.

(3) Unit hydrograph parameters and loss rates were adopted for each subbasin on the basis of a regional analysis.

(4) Routing criteria were developed.

(5) Rainfall distributions were determined for 2-year, 10-year, 25-year, 50-year, 100-year and Standard Project Storms by procedures described in Chapter 4 and Appendix.

(6) Depth-area rainfall data were computed for each of the hypothetical storms.

(7) A HEC-1 model was developed.

(8) Discharge hydrographs at each index station for the hypothetical storm events were computed with the basin HEC-1 rainfall-runoff model.

(9) A peak discharge-frequency curve was developed for the stream gaging station by procedures from the U.S. Water Resources Council Guidelines for Determining Flood Flow Frequencies (WRC, 1976) using regional gage data.

(10) The resulting frequency curves for the gage, one based on recorded flows and the other on the HEC-1 results, were compared and loss rates as previously adopted in the basin model were adjusted until the two frequency curves coincided.

(11) A regional frequency study of annual flood peaks of nearby hydrologically similar watersheds was performed. Results were evaluated on the basis of a similar study for the Upper Delaware and Hudson River Basins.

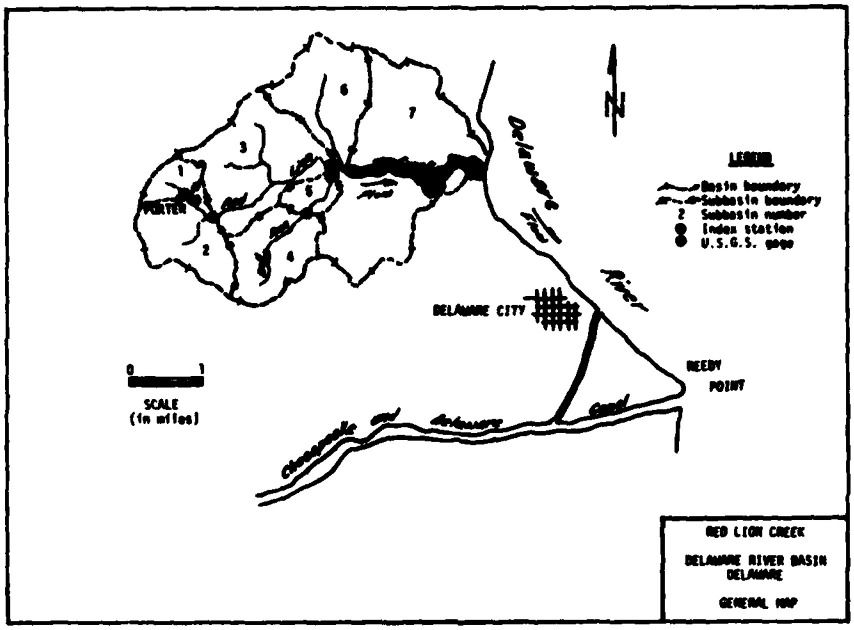
10.2 Basin Description

Red Lion Creek Basin is in northern Delaware just north of Delaware City and about 10 mi southwest of Wilmington. It has a total area of about 10 sq mi. Because the stream is a tributary to the Delaware River, stages in the lower reaches are affected by tidal conditions. Basin elevations range from sea level at the confluence with the Delaware River, to 80 ft, at the river's source. The total length of the main watercourse is about 5.4 mi. Stream slopes are very small in the lower reaches; they increase to about 10 ft per mi in the middle reaches and to 50 ft per mi in the uppermost reaches.

The study area is shown in Figure 10.1. The index stations where discharge-frequency information was requested are designated on the map.

10.3 Unit Hydrograph Analysis

Multiple regression analysis was used in a previous study to correlate t_p (Snyder's standard lag) and various physiographic characteristics of selected drainage basins in southeastern Pennsylvania (Hydrologic Engineering Center, 1976d).



10.1. Red Lion Creek Basin Study Area

This relationship was of the form:

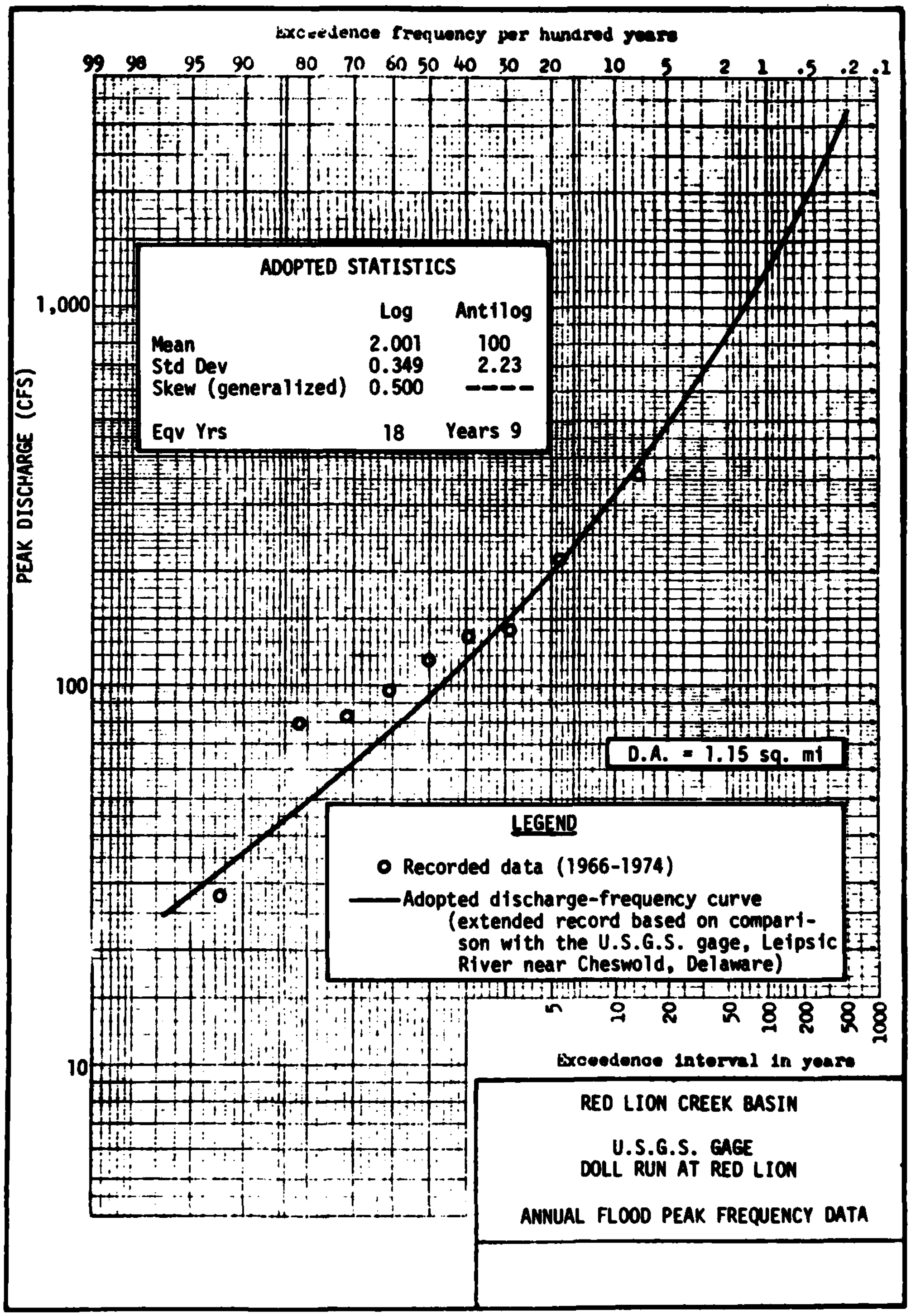
$$t_p = C_t \, (L \, L_{CA})^{0.30} \quad \ldots\ldots\ldots\ldots \quad (10.1)$$

The value of C_t for Red Lion Creek was computed from selected values of t_p for the nearby Christina River Basin (Stottler, Stagg and Associates, 1975). The t_p values used were selected on the basis of physical similarities between the gaged subareas of the Christina River Basin and Red Lion Creek. The resulting relationship for the Red Lion Creek Basin is

$$t_p = 1.55 \, (L \, L_{CA})^{0.30} \quad \ldots\ldots\ldots\ldots \quad (10.2)$$

10.4 Frequency Analysis

A crest-stage gage is located on Doll Run (see Figure 10.1). Records of *maximum* discharge are available for the water years 1966 through 1974 and recorded peak flows are plotted in Figure 10.2 according to the plotting formulation of Statistical Methods in Hydrology (Beard, 1962). Because of the short-term record at this gage (Doll Run at Red Lion, Delaware), numerous two-station comparisons were made in an effort to extend the record at this site. The only nearby long-term record that satisfied the recommended requirements given in Appendix 7 of Guidelines for Determining Flood Flow Frequency (WRC, 1976), is the USGS gage on the Leipsic River near Cheswold, Delaware. The available record at that station is 31 years (1944-1974).



10.2. Annual Flood Peak Frequency Data

According to the guidelines, if the following relationship is satisfied, then adjustment of the logarithmic mean and standard deviation of the short record is recommended:

$$r \geq 1.00 - 0.05\,(N_1 + N_2)/N_1 \quad \text{. (10.3)}$$

where

r = correlation coefficient between the logarithms of the flows from the short record and the logarithms of the flows from the long record during the concurrent period

N_1 = number of years when flows were concurrently observed at the two sites

N_2 = number of years when flows were observed at the longer record site but not observed at the short record site.

In this case for N_1 of 9 years and N_2 of 22 years, the inequality (10.3) becomes

$$r \geq 0.828 \quad \text{. . . . . (10.4)}$$

The computed value of r is 0.875; therefore, the logarithmic mean and standard deviation of the short record were respectively adjusted to 2.001 and 0.349, according to relationships contained in the guidelines. A generalized skew of 0.500 was adopted on the basis of results from a regional frequency study for the Delaware and Hudson River Basins (Hydrologic Engineering Center, 1974). The adopted statistics and the resulting discharge-frequency curve for the USGS gage, Doll Run at Red Lion, are shown in Figure 10.2.

10.5 Hypothetical Storms

Storms of various frequencies were developed for the basin under investigation. The storms selected were the Standard Project Storm (SPS) and those with recurrence intervals of 100, 50, 25, 10 and 2 years. The Standard

Project Storm (SPS) rainfall depth and distribution for the study area was developed with procedures from EM 1110-2-1411 (US Army Corps of Engineers, 1965). The 200-sq mi, 24-hour precipitation index for the basin location is 10.7 inches. This was adjusted to a 10-sq mi area, and then, using a transposition coefficient of 1.0, the SPS rainfall distribution was developed for a tabulation interval of 1 hour.

The 100-, 50-, 25-, 10-, and 2-year storms were developed by procedures described in the Appendix. Average point rainfall depths were taken from the isopluvial maps for the study area location. The depths are tabulated in Table 10.1 for each storm for durations from 1 hour to 24 hours. A rainfall distribution similar to that for the SPS was used in which the hour of greatest precipitation is preceded by the second greatest and followed by the third greatest. The rainfall depth-area data developed for the stream system procedure of HEC-1 are given in Table 10.2. This procedure automatically accounts for decreasing amounts of basin average precipitation with increased basin size (see Addendum 2, HEC-1 Flood Hydrograph Package; HEC, 1973a).

10.6 Hydrologic Basin Model

A hydrologic basin model was developed for this area using HEC-1. The area was subdivided such that flows would be computed at the various index stations indicated on Figure 10.1 and at the gaging station. Snyder's standard lag, t_p, was computed for each of the subbasins using equation (10.2). A C_p value of 0.65 was chosen based on the values for the Christina River Basin (computed values of t_p are given in Table 10.3). Channel routing coefficients were determined for the Muskingum routing technique. The Muskingum K's were estimated by the following relationship:

$$K = L/V \quad \text{(10.5)}$$

where

K = estimated reach travel time in hr

L = routing reach length in mi

V = estimated average reach velocity in mi per hr computed using Manning's equation

TABLE 10.1

AVERAGE POINT RAINFALL DEPTHS
FOR DURATIONS FROM 1 HOUR TO 24 HOURS AND
RETURN PERIODS FROM 2 TO 100 YEARS[1]

Duration (hours)	Return Period				
	100-Year	50-Year	25-Year	10-Year	2-Year
	Rainfall depth in inches				
1	3.35	3.00	2.70	2.40	1.55
2	4.15	3.65	3.20	2.80	1.85
3	4.50	4.05	3.60	3.10	2.05
4	5.25	4.75	4.20	3.65	2.40
5	6.30	5.60	5.15	4.35	2.80
6	7.30	6.50	5.75	5.15	3.25

[1]Data taken from the isopluvial maps for the study area in Technical Paper No. 40, Rainfall Frequency Atlas of the United States, Dept. of Commerce, Washington, D.C., May 1961.

TABLE 10.2

24-HOUR DEPTH-AREA RAINFALL DATA FOR HYPOTHETICAL STORMS

Drainage Area (sq. mi.)	Rainfall in Inches					
	SPF	100-Year	50-Year	25-Year	10-Year	2-Year
1	12.79	7.30	6.50	5.75	5.15	3.25
2	12.73	7.26	6.47	5.72	5.12	3.23
10	12.63	7.21	6.42	5.68	5.09	3.21
20	12.36	7.13	6.35	5.62	5.03	3.18

TABLE 10.3

SELECTED DATA IN THE VICINITY OF RED LION CREEK USED IN THE ANALYSIS RELATING t_p AND PHYSIOGRAPHIC CHARACTERISTICS OF A DRAINAGE BASIN[1/]

	Gage Location	Drainage Are (sq. mi.)	t_p (hours)	C_p	(miles)	L_{CA} (miles)
1	West Branch of Brandywine Creek @ Coatesville, Pa.	45.8	6.32	0.63	17.0	9.0
2	West Branch of Brandywine Creek near Honey Brook, Pa.	18.7	6.49	0.65	7.0	3.5
3	White Clay Creek near Newark, Del.	87.8	8.39	0.76	21.5	10.0
4	Christina River near Coochs Bridge, Del.	20.5	5.70	0.69	11.0	5.2
5	White Clay Creek above Newark, Del.	66.7	7.62	0.76	15.3	9.9
6	Little Mill Creek near Elsmere, Del.	6.7	2.34	0.55	4.5	2.2

1/ Adopted values of t_p and C_p taken from Appendix A of the preliminary hydrologic study for Christina River Basin, Pennsylvania.

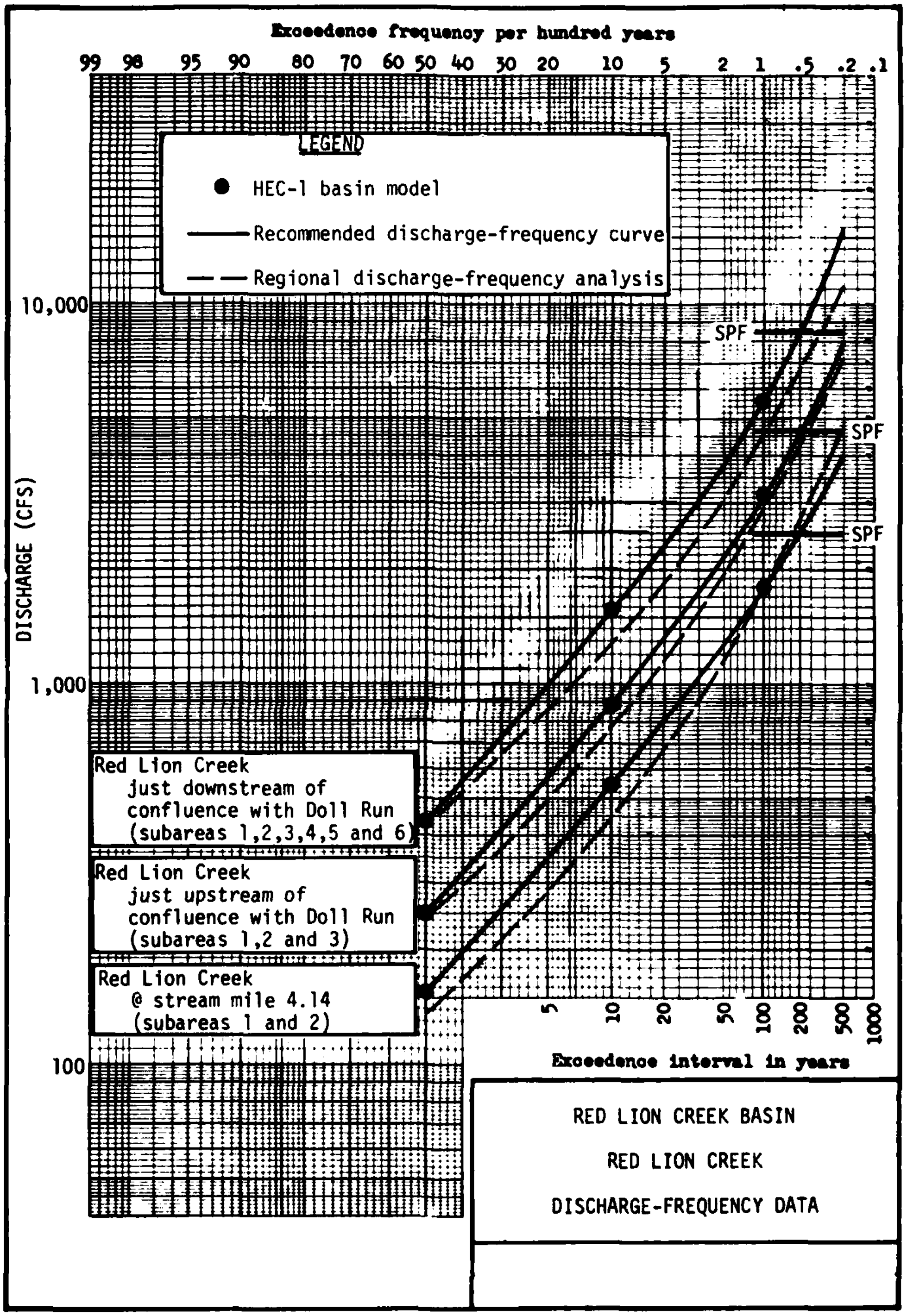
ıe starting discharge adopted for each subbasin was 2.0 cfs/sq mi ;tent with values for the hydrologically similar Christina River , Recession discharge was begun at a value of 10 percent of the peak rge, and RTIOR was set equal to 2.50. Loss rate parameters ranging .6 to 1.0 in. initial volume and 0.08 to 0.40 in. per hr constant, ing on the magnitude of the storm event, were used in the preliminary is.

ıe basin model was run under the preceding conditions for each ›tical storm event. Peak flows as computed at the USGS gage were ›d with flows obtained from the adopted discharge-frequency curve of 10.2. The initial loss rates were adjusted until the computed peak approximated the respective flows obtained in the frequency analysis. sults of the HEC-1 basin model and the adopted discharge-frequency for the gage site are shown in Figure 10.3. (To minimize effort and n subsequent analyses only the discharges resulting from the 2-year, r, 100-year and Standard Project Storm events were computed).

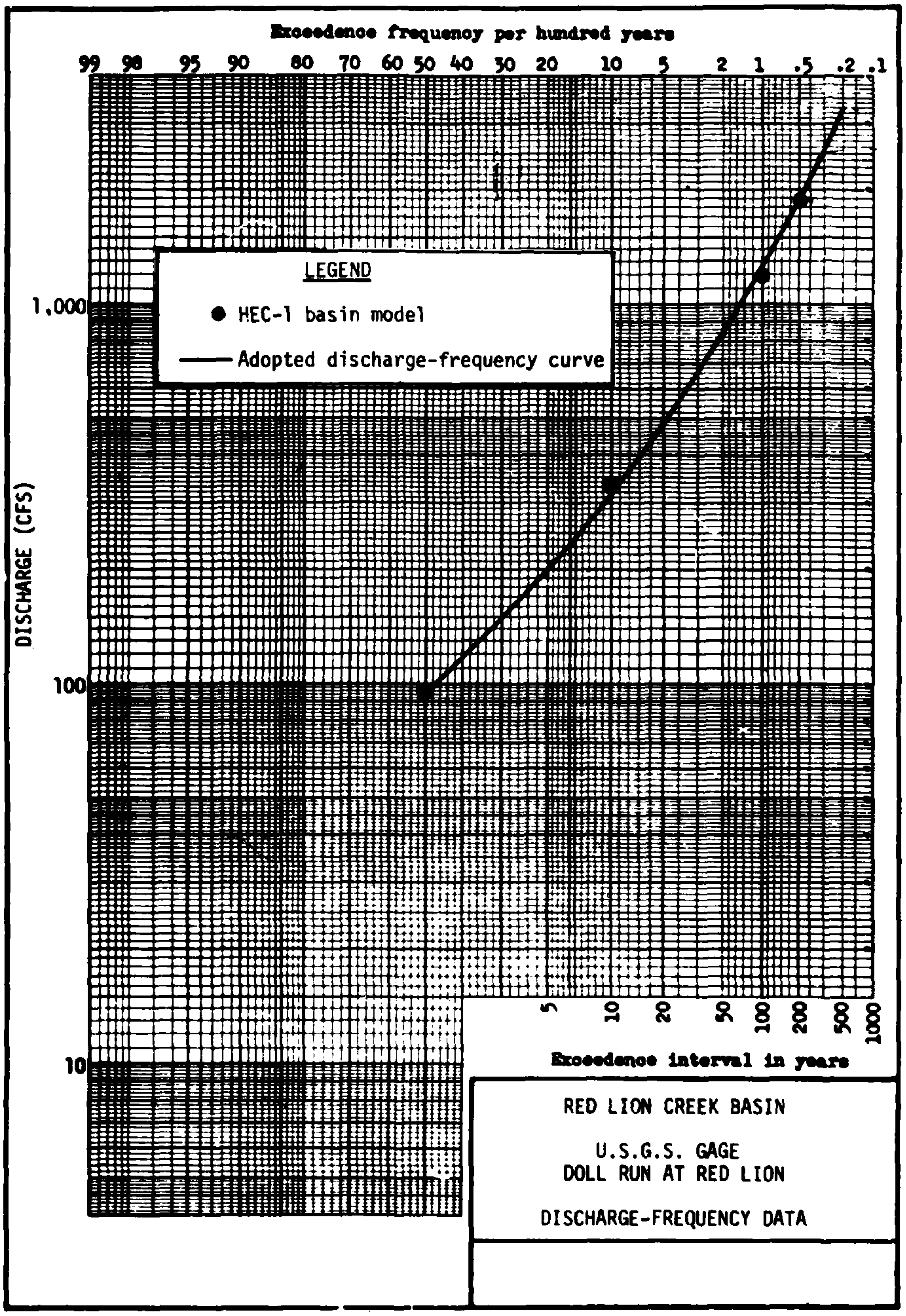
igure 10.4 gives plots of recommended peak discharges for the ›tical flood events at various index stations. The Standard Project has been represented by a horizontal straight-line segment in 10.4 because the frequency of this event cannot be stated accurately; t, it is not possible even to give it the same frequency at various stations within a study area. However, the range of probable ıcies for the SPF does offer an independent check for frequency curve ›ment. Figure 10.5 shows a plot of peak discharge versus drainage area : as a guide in computing the discharge at other points in the basin.

<u>tegional Discharge-Frequency Analysis</u>

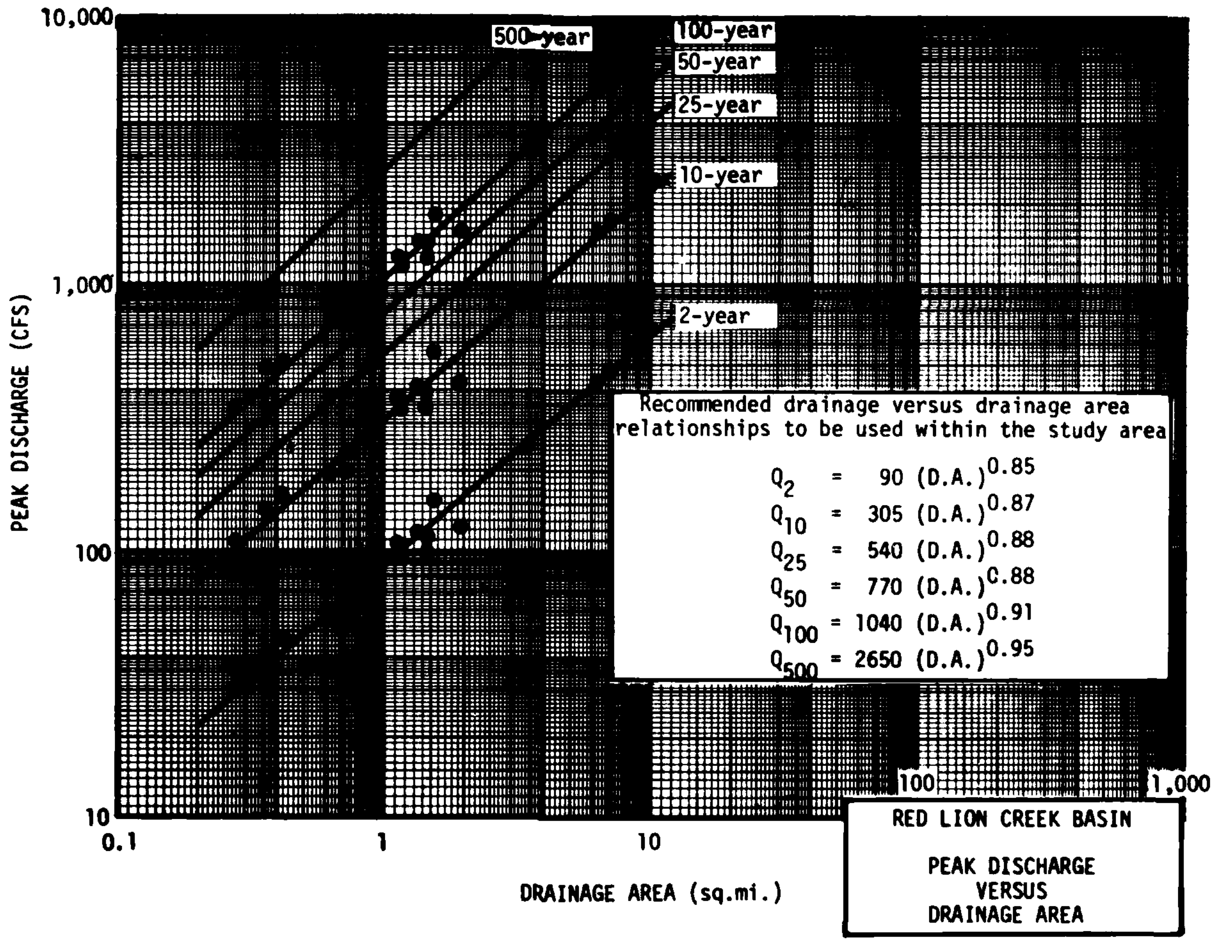
:ocedures for estimating discharge-frequency curves for ungaged areas ıed in the report <u>Regional Frequency Study, Upper Delaware and Hudson Basins, New York District</u>, (Hydrologic Engineering Center, 1974) were ›d in this study. Various basins considered hydrologically similar to udy area and within a 75-mile radius were analyzed according to .a contained in <u>Guidelines for Determining Flood Flow Frequency</u> (WRC, The computed statistics for the systematic record of each subbasin



10.3. Discharge-Frequency Data



10.4. Peak Discharge for Hypothetical Events



10.5. Peak Discharge vs. Drainage Area

are given in Table 10.4. Two-station comparisons were made only within this group, and short-term records were extended where the guideline conditions were satisfied. Records were also adjusted for low outliers, high outliers, and historical data.

Records were adjusted for low outliers if the following relationship was true:

$$\left|\frac{X_n - \bar{X}}{S}\right| > (2.5 + 1.2 \log (N/10)\ )\ (1.0 - 0.4\bar{G}) \quad \ldots\ldots\ldots (10.6)$$

in which

X_n = the logarithm of the lowest value in the sample of N annual flood peaks

$\bar{X}$ = mean logarithm of the N events

S = standard deviation of the logarithms of annual flood peaks

$\bar{G}$ = generalized skew coefficient

In this particular case, the generalized skew coefficient, $\bar{G}$, is 0.5; therefore the inequality (10.6) reduces to

$$\left|\frac{X_n - \bar{X}}{S}\right| > 2.0 + 0.96 \log (N/10)\ ) \quad \ldots\ldots\ldots (10.7)$$

Also, the weighted skew, G, was computed for records of 25 to 100 years by the equation

$$G = \frac{N - 25}{75} G_s + \left[1 - \frac{N - 25}{75}\right] \bar{G} \quad \ldots\ldots\ldots (10.8)$$

where G_s is the computed station skew.

For a $\bar{G}$ of 0.5, equation (10.8) becomes

$$G = \frac{N - 25}{75} (G_s - 0.5) + 0.5 \quad (10.9)$$

The generalized skew was used for records of 25 years and less. The adjusted (adopted) statistics are tabulated in Table 10.4. Map coefficients for the mean log of the annual peaks, C_m, and standard deviation, C_s, were computed from the adopted statistics and the following regression equations:

$$\log (Q_m) = C_m + 0.87 \log (DA) \quad (10.10)$$

where

Q_m = geometric mean of the annual flood peaks, cfs

C_m = a map coefficient for the mean log of the annual peaks

$$S = C_s - 0.05 \log (DA) \quad (10.11)$$

where

S = standard deviation of the logarithms of the annual flood peaks

C_s = a map coefficient for the standard deviation

The computed map coefficients are shown in Figures 10.6 and 10.7. The generalized trend of the map coefficients indicates that values of 1.95 and 0.35 for C_m and C_s, respectively, are reasonable for the Red Lion Creek Basin. Discharge-frequency curves (computed by procedures contained in the Delaware River Basin Study report, HEC, 1974), based on these particular values and using expected probability and an estimated equivalent length of record of 15 years, are shown in Figure 10.4 for comparison.

TABLE 10.4

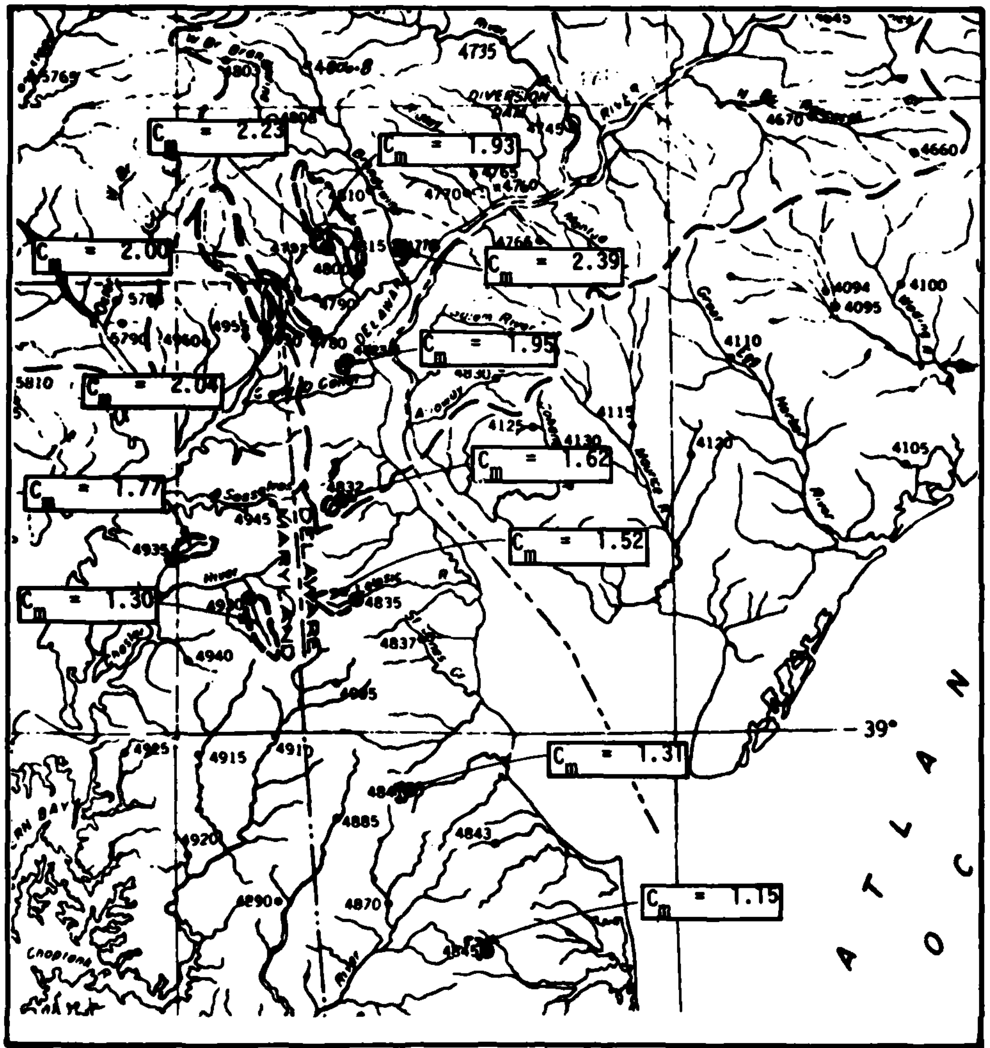
DISCHARGE-FREQUENCY STATISTICS FOR STREAMS IN THE VICINITY OF RED LION CREEK

Gage No.	Gage Name	D.A. (sq.mi.)	Period of Record (water year)	Computed Statistics[1] Mean	Std. Dev.	Skew	Adopted Statistics Mean	Std. Dev.	Skew	C_m	C_s
				---------- log	----------		---------- log	----------			
4778	Shellpot Creek @ Wilmington, Del.	7.46	1946-74	3.152	0.272	0.728	3.152	0.272	0.512	2.39	0.32
4780	Christina River @ Coochs Bridge, Del.	20.5	1943-74	3.183	0.123	0.784	3.183	0.123	0.527	2.04	0.19
4792	Mill Creek @ Hockessin, Del.	4.19[2]	1966-74	2.762	0.248	1.407	2.762	0.248	0.500	2.23	0.28
4800	Red Clay Creek @ Wooddale, Del.	47.0	1944-74	3.380	0.164	0.518	3.380	0.164	0.501	1.93	0.25
4823.1	Doll Run @ Red Lion, Del.	1.15	1966-74	2.053	0.308	-0.436	2.001	0.349	0.500	1.95	0.35
4832	Blackbird Creek @ Blackbird, Del.	3.85	1951-74	2.126	0.321	0.299	2.126	0.321	0.500	1.62	0.35
4835	Leipsic River near Cheswold, Del.	9.35	1944-74	2.366	0.325	0.796	2.366	0.325	0.524	1.52	0.37
4841	Beaverdam Branch @ Houston, Del.	2.83	1958-74	1.707	0.231	0.045	1.707	0.231	0.500	1.31	0.25
4845	Stockley Branch @ Stockley, Del.	5.24	1943-74	1.777	0.179	-0.086	1.777	0.179	0.445	1.15	0.22
4930	Unicorn Branch near Millington, Md.	22.3	1949-74	2.475	0.281	0.006	2.475	0.281	0.500	1.30	0.35
4935	Morgan Creek near Kennedyville, Md.	10.5	1952-74	2.655	0.394	1.090	2.655	0.394	0.500	1.77	0.45
4950	Big Elk Creek @ Elk Mills, Md.[3]	52.6	1932-74	3.491	0.232	0.373	3.499	0.243	0.515	2.00	0.33

[1] Computed statistics based on the systematic record for each station.

[2] Drainage area taken as 4.04 sq. mi. since 0.15 sq. mi. probably noncontributing.

[3] Adjusted for historic data - 18,000 cfs in 1884.

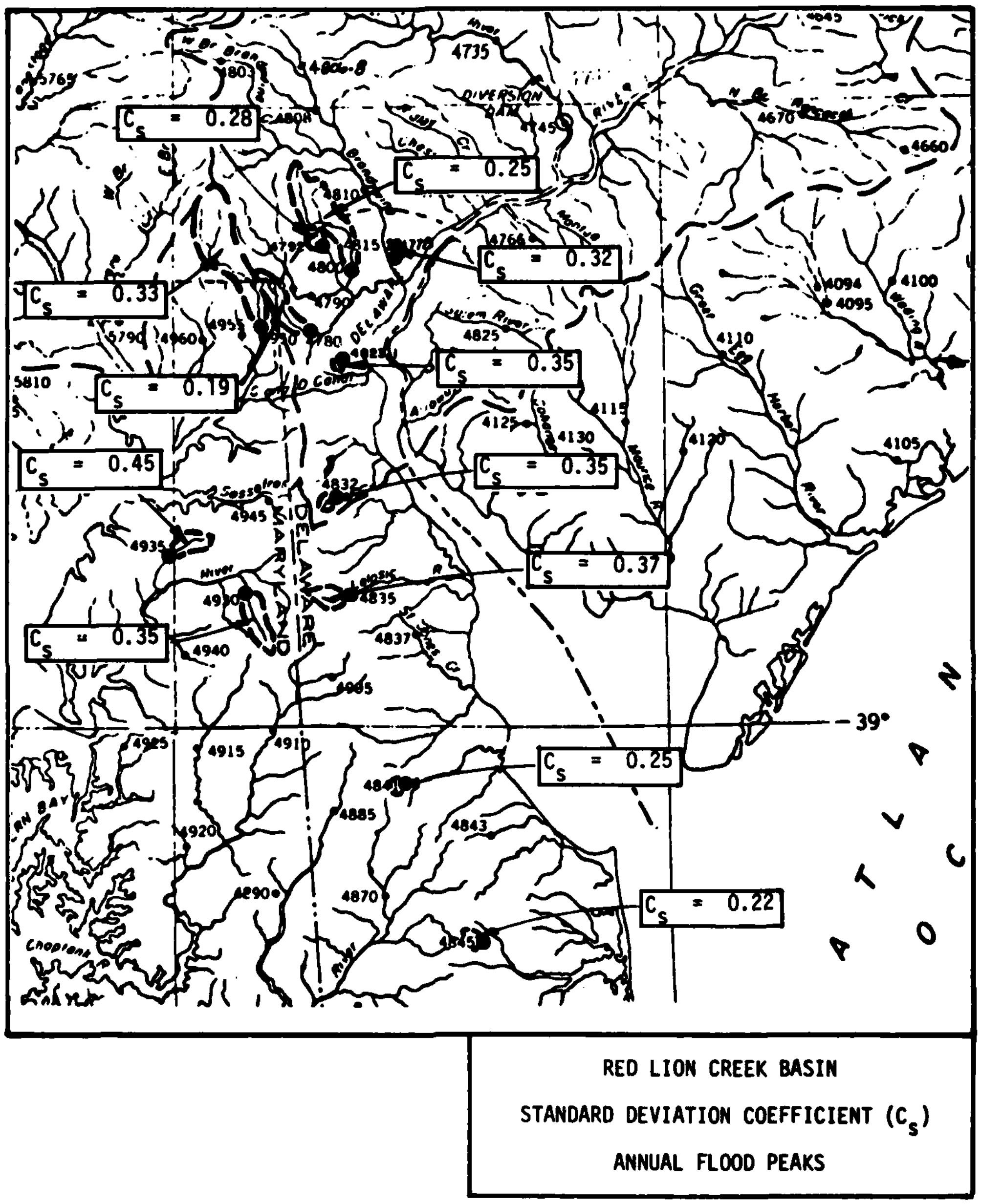


RED LION CREEK BASIN

MEAN LOG COEFFICIENT (C_m)

ANNUAL FLOOD PEAKS

10.6. Mean Log Coefficient C_m for Annual Flood Peaks



10.7. Standard Deviation Coefficient C_s for Annual Flood Peaks

CHAPTER 11

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APPENDIX

PROCEDURES FOR DEVELOPING SYNTHETIC PRECIPITATION DATA

A.1 Introduction

Synthetic or hypothetical storms are frequently used by the Corps of Engineers to generate synthetic flood events for evaluation of existing flood conditions and for study of the effects of flood-mitigation components. The degree of dependency on synthetic flood events during a flood evaluation depends generally on the number of gages and the length of hydrologic record available. For reservoir studies there is typically at least one long-record stream gage near the dam site, and thus the storage required for reservoir flood control is usually computed by period-of-record simulation techniques using recorded data. Channels, levees, and other local protection projects are more likely to be sized by synthetic storm-flood analyses since these projects often extend over many miles of the stream, and it is rare that gage information would be available throughout the reach.

Hypothetical storms are also used to ensure that rare storm-flood events are included in an overall assessment of the proposal. Since large storms that cause major floods are infrequent, the available runoff record quite possibly may not reflect the occurrence of these rare events. Analyses of major reservoirs, although relying mainly on recorded stream data, also require the use of synthetic storms (Standard Project and Probable Maximum Storms) to evaluate the safety of major dam components, such as the spillway and outlet works, and guard against overtopping of the dam. Synthetic precipitation data are also used for analyses of flooding throughout a basin, testing of the hydrologic effects of alternative land uses and flood plain regulations, and evaluation and design of flood control components. This is usually done with a model of the watershed to simulate the rainfall-runoff process, with the rainfall specified by hypothetical storms.

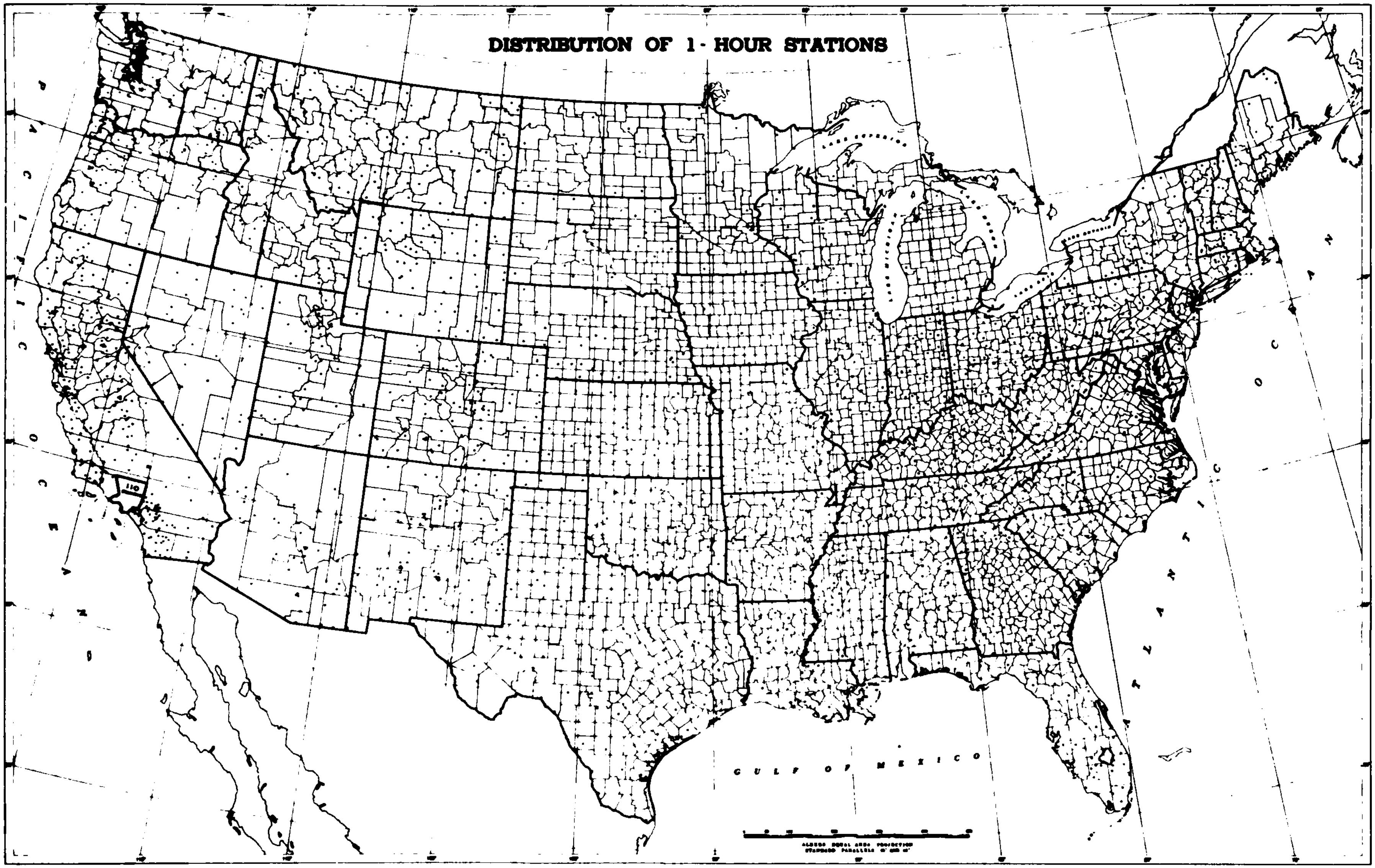
A.2 Sources of Storm Data

The primary sources of hypothetical storm information for the United States are various technical publications (TP) of the National Weather

Service (NWS) and hydrometeorological reports (HR) of the National Oceanic and Atmospheric Administration (NOAA), such as references A.1 through A.16. All publications for development of hypothetical frequency storms feature generalized isopluvial rainfall maps and/or regression equations. Other methods, such as statistical analysis of nearby long-record rain gages to derive hypothetical storms of particular frequencies in lieu of the NWS and NOAA procedures, are used extensively in some parts of the United States but not discussed here.

The geographical areas to which these publications apply is shown in Figure A.1. References A.1, A.2 and A.3 are used for the 35 states east of the Rocky Mountain area which are essentially free of significant orographic effects. The particular publication to be used depends on the storm duration under examination. The 13 mountain states containing the Rocky Mountains and those areas to the west are covered by references A.3., A.4, and other site-specific publications. The procedures described in all the NWS publications are based on statistical evaluations of long-term rainfall-gage records in a region. These evaluations include estimates of the frequency of accumulated rainfall-depth versus storm duration at each rain gage. Rainfall maps were made from these depth-duration values, and isopluvial lines were drawn on these maps to define constant rainfall-depth relationships through a region for a specific storm duration. An isopluvial map is shown in Figure A.2. Each of the NWS publications gives a detailed discussion of the derivation of the rainfall-frequency depth-duration relationships. It is strongly recommended that users of NWS data thoroughly read the pertinent sections and be familiar with the applications and limitations of the NWS material.

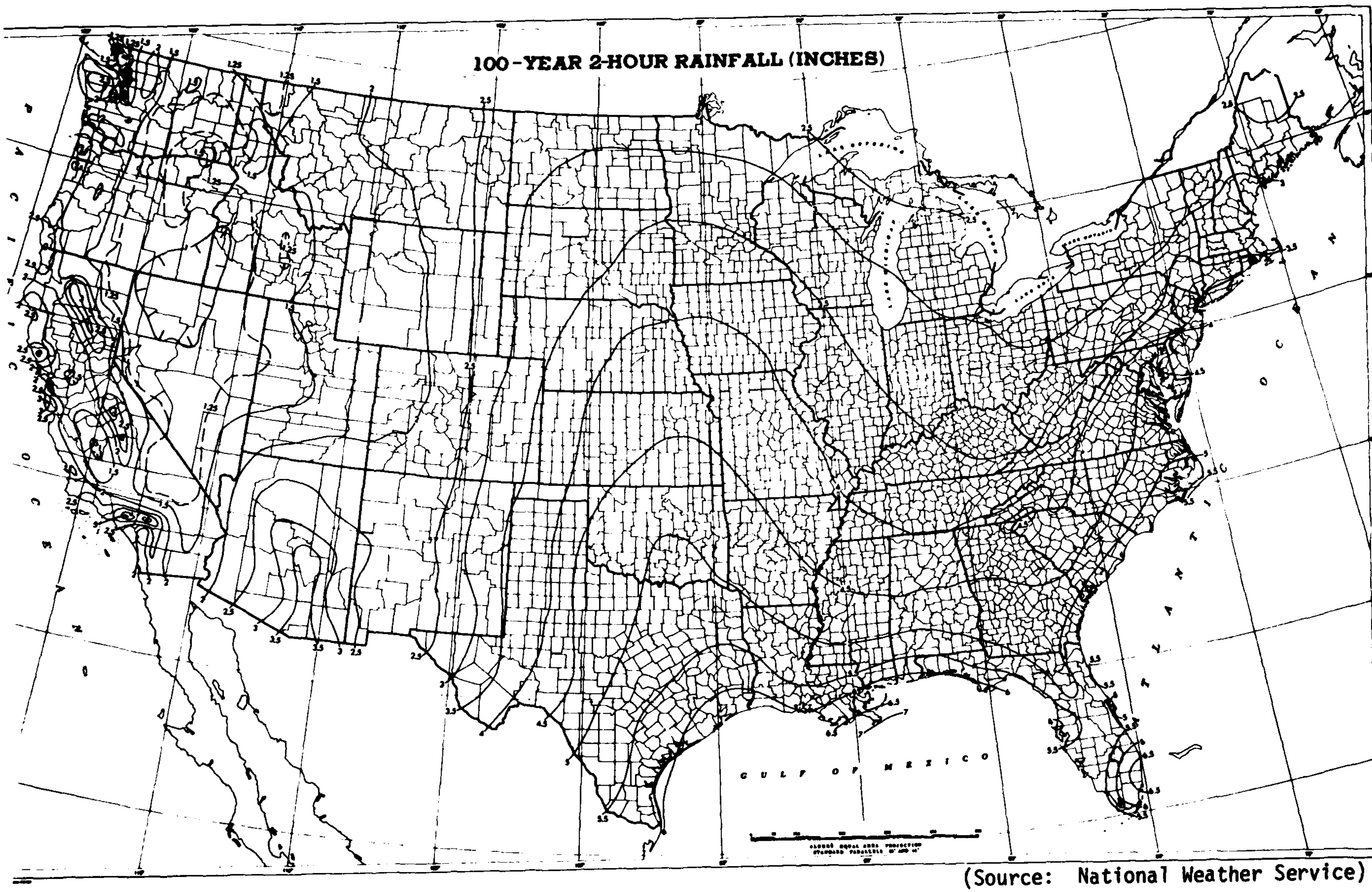
A hypothetical storm developed from NWS data is also referred to as a "balanced storm," because a consistent depth-frequency relation is used for each peak duration interval of the storm. That is, for a hypothetical 100-year return period 48-hour-duration storm, the rainfall depths for the peak 30-minute, 1-hour, 6-hour, 24-hour or other peak period would each be equal to the 100-year depth for that duration. This consistent frequency-depth-duration relationship throughout a storm would not occur in nature, because of the randomness of the rainfall events. The balanced storm concept, however, does allow for logical construction and arrangement of a storm event for a particular return period. Balanced storms are discussed in more detail later in this section.



Distribution of 1-hour stations.

(Source: National Weather Service)

A.1. Distribution of Rainfall Stations for NWS Storm Data



(Source: National Weather Service)

A.2. Typical Isopluvial Map

The Standard Project Storm (SPS) is used primarily by the Corps of Engineers and is described in reference A.9. It is treated in detail in paragraph A.5. Development of the SPS for states west of the Rocky Mountains requires site-specific criteria, which are not discussed here.

The Probable Maximum Storm (PMS) is developed primarily from NOAA and NWS criteria, some of which is given in references A.10 through A.18. The area east of the Rocky Mountains is covered by reference A.16. Paragraph A.6 of this report describes the derivation of the PMS.

A.3 Hypothetical-Frequency-Storm Derivation

Development of a storm from NWS data is straightforward and systematic. The individual performing the study must: 1) establish the appropriate storm duration and the time interval for subdividing the storm rainfall, 2) extract the rainfall values from NWS publications for his area of interest, 3) make adjustments to the rainfall depth for size of drainage area if needed, 4) adjust for partial to annual series (if required), 5) compute incremental rainfall amounts, and 6) arrange the storm rainfall increments in time. Each of these steps is described in the following paragraphs. Some example calculations to develop storm rainfall are given in Section A.4.

Storm Duration. Before constructing any hypothetical event (including the SPS and PMS), one must estimate two storm parameters: total duration and time interval for each rainfall increment. Both parameters must reflect the type and size of the drainage area being examined, the type of basin features one intends to analyze, and the location of these features. The total duration of the hypothetical storm is directly related to the time of concentration of the watershed (the travel time from the upper portions of the watershed to the most downstream point of interest). For example, if the estimated travel time is 14 hours (determined from actual records or by computation) from the watershed boundary to the lower limits of the study area, the storm duration must be at least 14 hours and preferably more. For most applications, the duration would be set to an even day (24 hours). Since a storm duration of less than 14 hours would not allow all portions of the drainage basin to contribute direct runoff to the outlet simultaneously during the course of the storm, the peak discharge at the basin outlet would

not be reflective of the rainfall event if the storm duration were made less than 14 hours. Runoff from the lower portions of the basin from, say, a 6-hour event would have left the basin before the inflow from the upper portion reached the outlet. Therefore, a minimum storm duration should be selected at least equal to, and preferably well in excess of, the estimated travel time (time of concentration) at the downstream-most point of interest. This selected duration should be increased considerably if total volume of runoff as well as peak discharge is of importance in the study. Drainage basins having an unusually large amount of flood plain storage (wide flood plains and/or large areas of swamps) may require a storm of longer-duration to capture the attenuation effect of these large natural storage areas. Reservoir studies require long-duration events for full assessment of the reservoir flood storage needed. Therefore, a maximum storm duration of 10 days may be used even if the travel time to the reservoir site is only 14 hours. Total storm duration is normally taken as some increment of a 24-hour day (3, 4, 6, or 12 hours), or a multiple of a day (1 to 10 days).

Time Interval. Once the storm duration has been established, the time interval for subdivision of the total storm must be selected. The time interval must be small enough to accurately define the flood hydrograph (especially the peak); however, too small an interval will result in excess computations by the individual or the computer. The time interval will generally be established by the fastest peaking subarea of the overall basin model for which the peak discharge is required, i.e., for later use in developing water surface profiles, to evaluate the effects of a flood control component, etc. The time interval must be small enough to define the rising limb and peak for the hydrograph for this subarea. It has been found from past experience that a time interval that gives at least 3 points on the rising limb of the hydrograph prior to the peak provides an estimation of the peak discharge that is accurate enough for most work.

Extending the previous example of the selection of a storm duration of 24 hours based on a travel time of 14 hours to the outlet, one can now select a time interval of 4 hours (14 hours divided by 3 points prior to peak = 4.67 hours, rounded down to 4 hours). This would be an appropriate time interval if one were interested only in the peak discharge at the outlet. However, if

there is a subarea upstream for which a peak discharge estimate was also necessary, then the time interval for the entire drainage basin would be based on the requirements for this subarea. For example, if the time of concentration for this subarea is 70 minutes, the interval required would be 20 minutes (70 minutes/3 points prior to peak = 23.3 minutes, rounded down to 20 minutes). Note that the 24-hour storm would now be subdivided into 72 twenty- minute intervals to define the rainfall distribution for the basin, compared with the 6 four-hour intervals where only the discharge hydrograph at the outlet was needed. Only one time interval can be specified for a given HEC-1 simulation. This means that the smaller intervals must be carried throughout the hydrograph construction, routing, and combining operations within the HEC-1 model, and this greater number of intervals will require more effort and computer time. Extreme cases occasionally occur, such as a large drainage area with a few of its subareas very small in size. Although the storm duration may be 2 days or more to reflect travel time to the outlet, a time interval of 5 minutes may be required to accurately capture the peak discharge from the smallest subareas. It may be more economical to construct a separate HEC-1 model (most likely with a short-duration storm) for each small subarea requiring a 5-minute subdivision, thereby allowing the model of the large area to use multi-hour time increments for the longer storm duration.

Data Extraction from NWS Publications. The methodology for hypothetical storm development for the Western states will be described separately from that for the remainder of the United States. Procedures for developing hypothetical storms in Hawaii and Alaska are similar to procedures for the other states.

(a) Eastern and Central United States. Once the storm duration and computation time interval have been established, the rainfall depths for key durations and each desired return period are taken from the appropriate NWS publications. For a 6-hour storm duration and 15-minute increment, for example, the 2-year hypothetical storm data for the area of interest would be obtained from TP-40 and HYDRO-35. TP-40 gives isopluvial maps of the 2-, 3- and 6-hour duration 2-year-return-period total rainfall. The 30-minute and 1-hour maps in TP-40 have been superseded by the procedures given in HYDRO-35. By determining the location of the study area on each map, one can

select the 2-year rainfall depth for each of the three durations. Since durations of one hour and less are also needed, one must use the HYDRO-35 maps of 15- and 60-minute rainfall depth and repeat the extraction of the desired 2-year rainfall depths. A value of the 30-minute rainfall depth is also obtained by applying the equation given in HYDRO-35. At this point, all available rainfall-depth information for a 6-hour-duration 15-minute-interval storm has been extracted from the NWS publications. These six values (15- and 30-minute, 1-, 2-, 3-, and 6-hour rainfall depths) will be used in the examples which follow.

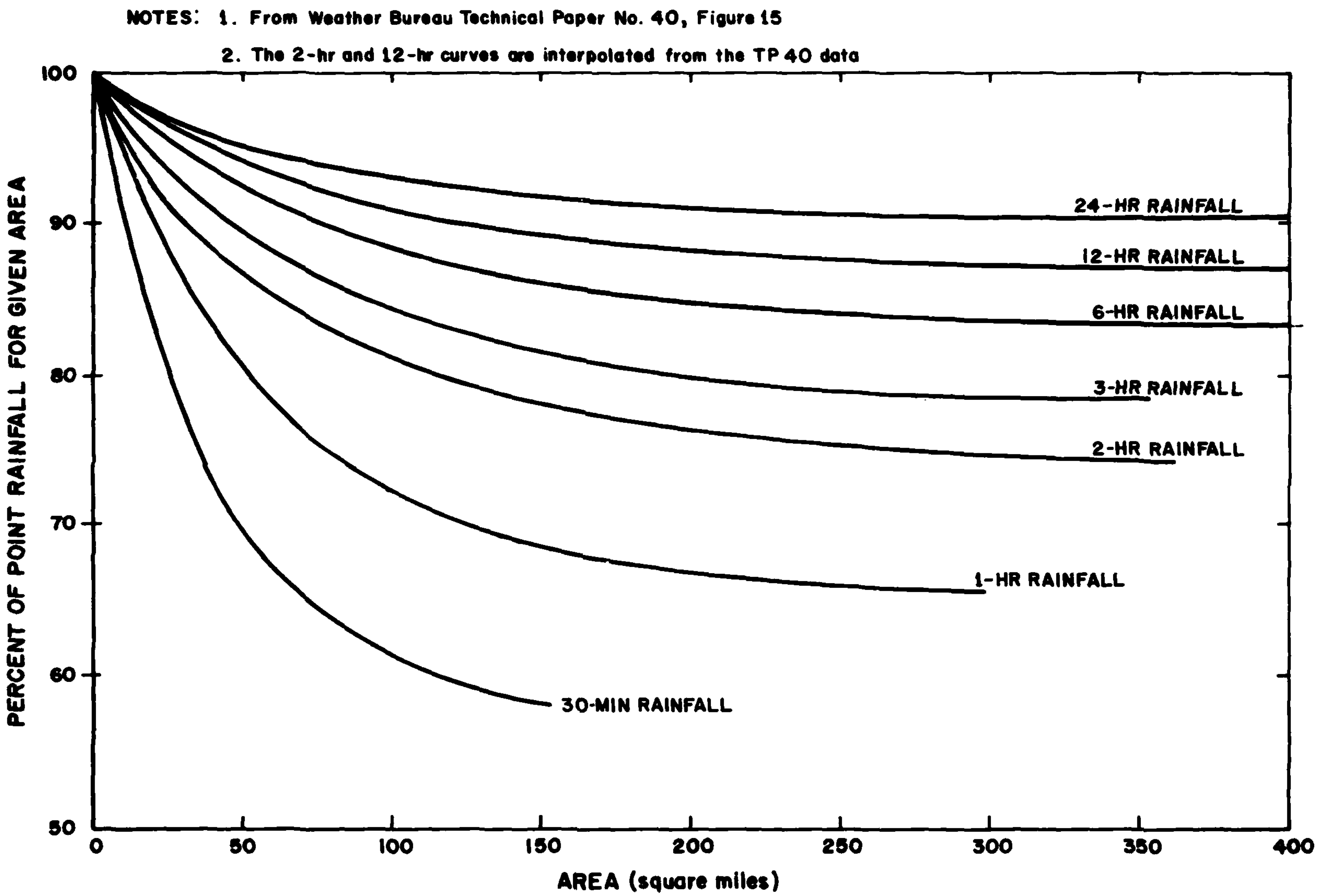
(b) Western United States. Procedures for extracting basic rainfall data from NOAA Atlas 2 for the Western states are significantly different from procedures for the rest of the United States. Maps are available for only the 6- and 24-hour-duration for various return periods. To determine a 6-hour duration 15-minute-interval 2-year-return-period storm, one would use the appropriate volume for the state in which the study area lies and extract the 6-hour and 24-hour rainfall depth at the location of the study area from the two maps. Although the total storm duration is only 6 hours, the 24-hour depth is usually needed to solve for the 1-hour-duration depths using the appropriate ratio and equation. Equations for the 2- and 3-hour-duration depths can be solved once the 1- and 6-hour depths are known. Durations of less than one hour are determined by multiplying the one hour-depth by various ratios given in Atlas 2. For the example, the maps and equations in Atlas 2 would be used to extract the 15- and 30-minute, 1-, 2-, 3- and 6-hour rainfall depths.

Areal Adjustment. Regardless of which publications were used, one now has six rainfall-depth values that must be further modified by one or more adjustment factors. The first adjustment factor is applied to the rainfall data taken from the publications. These depths are "point rainfall depths"; that is, as measured at a rain gage, i.e., a single point. The hypothetical storm will be applied to a specific watershed having a defined drainage area. For example, the amount of rainfall from a particular return-period event over a (say) 50-square-mile area would not be the same as that at a point, but would be less. A storm cannot be as intense when spread over a large area as it can be over a single point. Although the rainfall depth for any finite drainage area will be smaller than the value at a point, the

adjustment is often not made unless the study area is more than 5 to 10 square miles. As seen in Figure A.3, the application of the adjustment factor for small areas results in rainfall values that are little different from the point values. When the drainage area is larger than about 10 square miles, the adjustment becomes significant, particularly for the 30- and 60-minute durations. For the example from the previous paragraph, Figure A.3 would be entered at the drainage area under study and adjustment values for the 30-minute and 1-, 3-, and 6-hour durations would be read from the y-intercept of the drainage area and the appropriate curve. These four factors would be plotted against point rainfall depth on semi-logarithmic paper. The 2-hour-duration factor can be read from the curve, and the curve can be extrapolated to obtain a 15-minute duration factor. Since the NWS publications provide no guidance for adjustments for durations of less than 30 minutes, extrapolation to shorter durations is subject to question. This difficulty should not arise often, however, since a time subdivision of 5 or 10 minutes would imply a short time of concentration and a small basin. If the basin is small, an areal adjustment is not significant.

The adjustment factor found for each duration is used to modify the corresponding rainfall depth for that duration by multiplying these two quantities. Once this step has been completed, all rainfall values have been adjusted for the particular drainage area being studied.

Partial-to-Annual Series Adjustment. The previously described rainfall amounts are for partial-duration series of rainfall values. Conversion of rainfall values to an annual series may be needed. This adjustment is applicable only to return periods with a frequency of 10 years or less. The rainfall depth-frequency curve is converted from one for which one or more events per year were used (called a partial-duration series, which is one in which all events above some selected base value are used) to one using only a single event each year (called an annual series, in which the single highest event each year is used, even though the second highest in some years may be greater than the highest in other years). For economic analysis of agricultural areas, it is possible that damages are sustained by flood events that have a probability of occurrence of more than once per year. Where several floods per year are causing significant damage, it would be desirable to use a partial-duration series and not make the adjustment. In many



AREA-DEPTH CURVES

A.3. Adjustment of Point Rainfall for Watershed Area

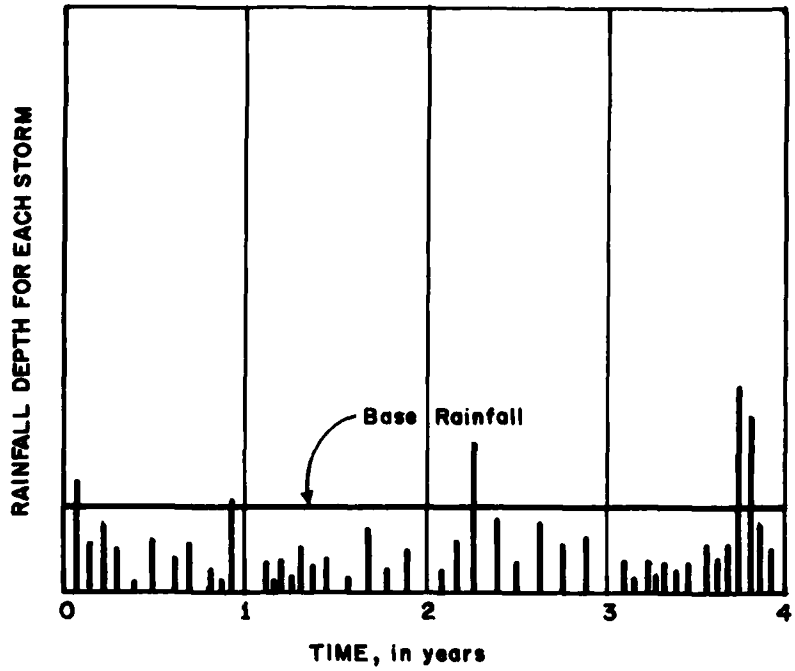
economic analyses, especially in urbanizing areas, these multiple floods (once- or twice-per-year events) do not cause significant damage. Therefore, the adjustment factor would be applied to prevent the use of biased (high) rainfall depths to determine the low end of the frequency curve. Figure A.4 illustrates the two series. The adjustment from partial-to-annual series is performed by multiplying the rainfall depths for each duration by the appropriate conversion factor. The conversion factors are as follows:

Series	Factor
2-year	0.88
5-year	0.96
10-year	0.99

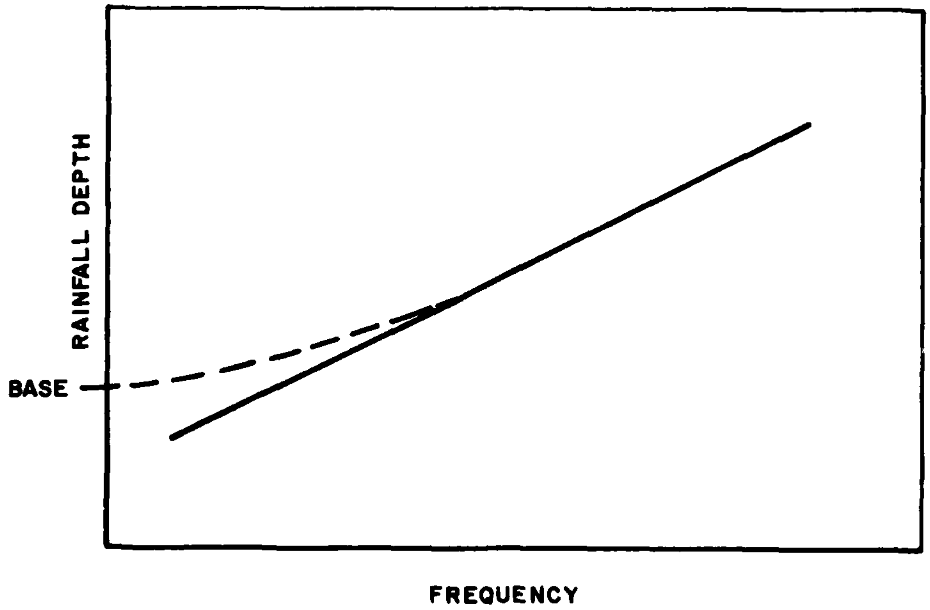
No adjustment is needed for rarer return intervals since the two curves coincide after the 10-year event. At this point, all adjustments have been made. The next step is to proceed through further breakdown and arrangement of the hypothetical storm.

Division into Incremental Values. One now takes the adjusted rainfall values for a particular storm (there are usually six values) and further subdivides these to arrive at a rainfall depth value for each time increment (for example, there will be twenty-four values for the 6-hour-duration 15-minute-interval case). This division into increments is usually performed by plotting the values of rainfall depth (in inches) versus duration (in minutes) on logarithmic paper, fitting a curve through these points, and then reading off accumulated depth values for each increment from the curve. Averaging the incremental change between the original points is usually a satisfactory alternative, since the depth-duration plot normally approximates a straight line after the first several values. Once an accumulated depth for each interval has been determined, the depths are incremented to compute that portion of the depth that occurred in each period.

Storm Arrangement. The final step in the storm definition is arrangement of the storm rainfall into a specific pattern. The pattern used most often by the Corps of Engineers is a "triangular" arrangement, with the peak period in the center of the storm. For our example, the peak 15-minute depth would be placed (assigned) to the thirteenth period of the twenty-four



a). Illustration of Partial and Annual Series



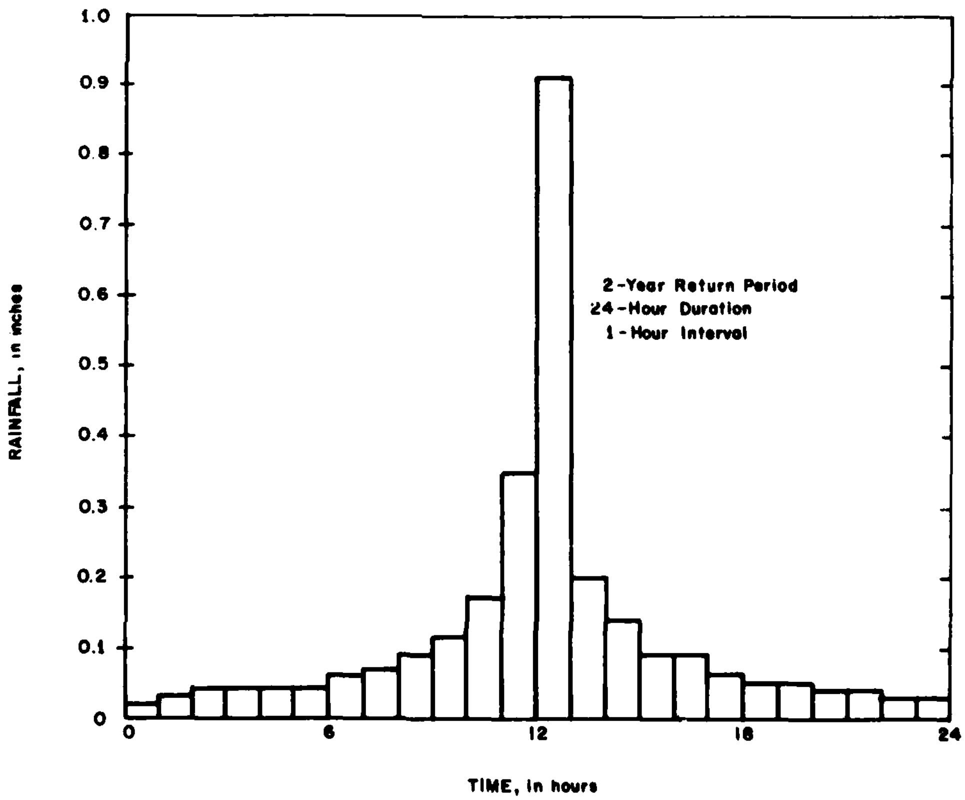
b). Comparison of Rainfall Depth vs. Frequency Curves for Partial and Annual Series.

A.4. Partial and Annual Rainfall Series

period storm sequence. The next-highest depth is placed just ahead of the peak (Period 12), the next highest depth just behind (Period 14), and so on until all 24 values are systematically arranged about the peak period.

If a storm with a duration longer than 24 hours is to be arranged, all 24-hour periods outside of the peak 24 hours can be represented by an average value for each 24-hour period. The rainfall increments cannot be moved outside the 24-hour period from which the increment was developed, however. Figure A.5 shows a sample arrangement for a twenty-four-hour rainfall.

The Standard Project Storm distribution can be used to develop an alternate arrangement of rainfall in time. However, although the SPS arrangement is sometimes applied to hypothetical-frequency storms, it was specifically derived for events much rarer than even the 100-year return-period event. Since its application will give a more severe arrangement than may be reasonable for a hypothetical-frequency event, estimates of peak discharge may be excessively high when the SPS arrangement is used.



A.5. Example of Arrangement of Hourly Rainfall Over a 24-Hour Period

mmary. The steps in developing a hypothetical storm rainfall include:

- . Determine total storm duration
- . Determine the time interval for subdividing the storm
- . Extract data from the appropriate NWS or NOAA publications
- . Adjust for area
- . Adjust for annual series, if necessary
- . Develop relation for accumulated depth versus time
- . Increment depths for each period
- . Arrange storm

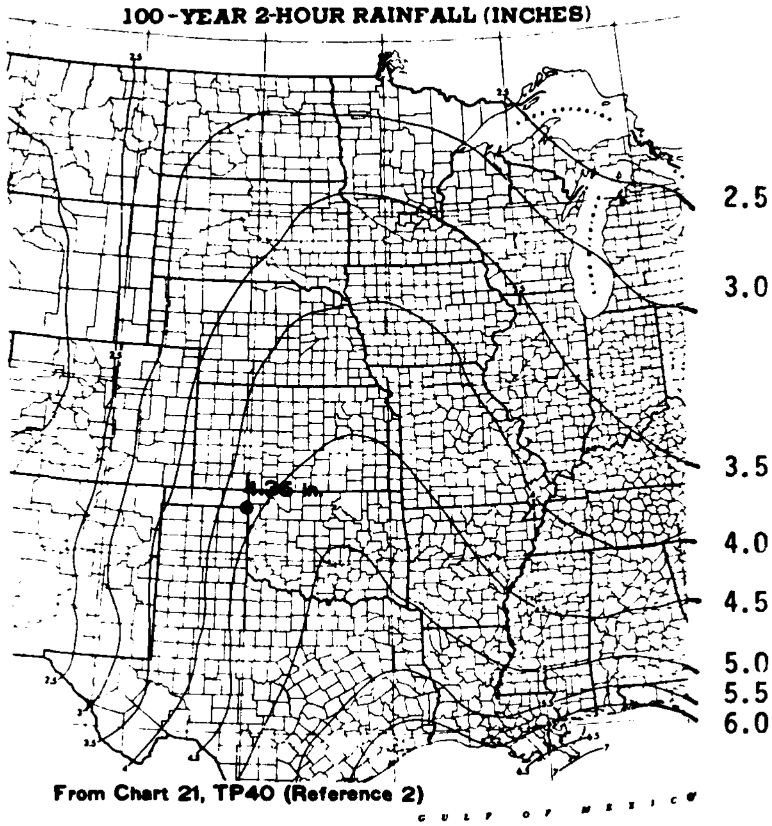
:ction A.4 gives examples of hypothetical-storm determination, and ı A.7 furnishes additional information on special problems in :tical-storm usage.

(amples

vo example problems are presented here, one for each of the two major)hical areas of the United States.

ıstern and Central U.S. Hydrologic evaluation of a 100-square-mile ıed located at the northeast corner of the Texas "panhandle" requires)ment and arrangement of the 100-year recurrence-interval hypothetical The total storm duration determined for the analysis is 12 hours with interval of 30 minutes.

) determine the properties of a 12-hour-duration 30-minute-interval in this region, both TP 40 and HYDRO-35 must be used. From TP 40, the or the 100-year return period are used to obtain rainfall depths for ak 2-, 3-, 6-, and 12-hour durations at the northeast corner of the panhandle. Values for the 15- and 60-minute rainfall depths are :d from the maps in HYDRO-35. For example, the value for the 100-year rainfall for the study area is obtained from TP-40 as shown in Figure The values found from TP-40 and HYDRO-35 are:



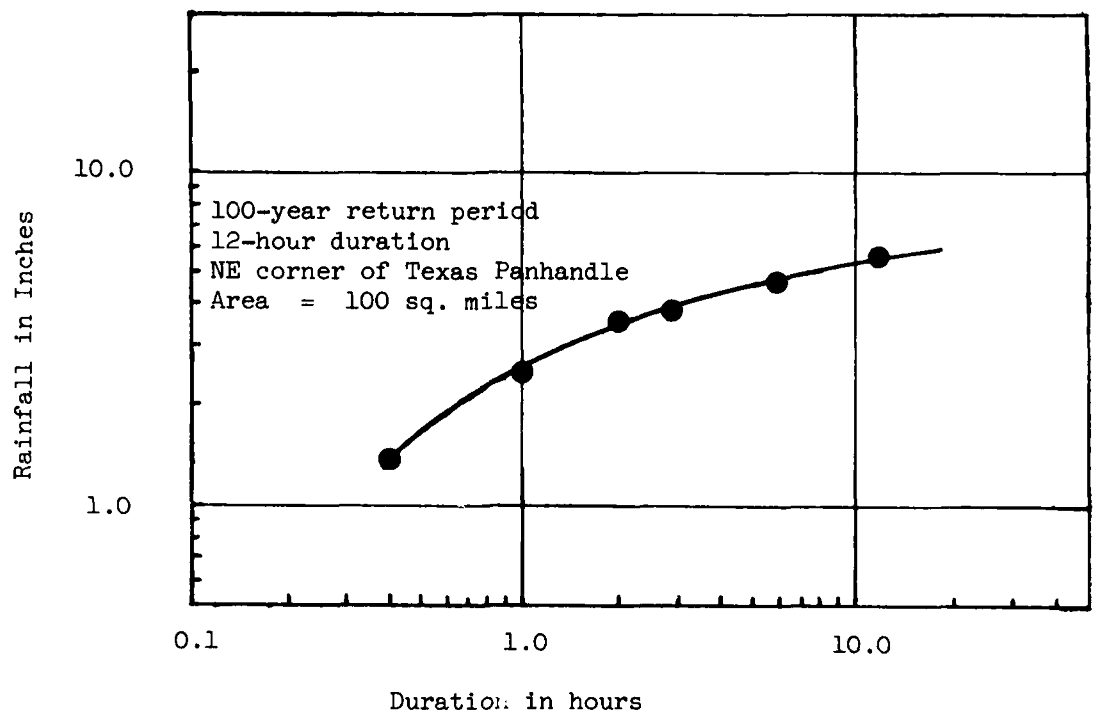
A.6. Rainfall Value for the Study Area
(Northeast Corner of Texas Panhandle)

Duration	15-min	60-min	2-hr	3-hr	6-hr	12-hr
100-year Rainfall	1.82 in.	3.46 in.	4.35 in.	4.65 in.	5.30 in.	6.00 in.

The 30-minute value is computed from Equation 7 of HYDRO-35 as follows:

$$
\begin{aligned}
\text{30-min value} &= 0.49 \times (\text{60 min-value}) + 0.51 \times (\text{15-min value}) \\
&= 0.49 \times (3.46) + 0.51 \times (1.82) \\
&= 2.62 \text{ inches}
\end{aligned}
$$

These values of accumulated point rainfall for various durations are shown in the second column of Table A.1. The 30-minute to 12-hour point values are next adjusted for a drainage area of 100 square miles. Adjustment values are taken from Figure A.3 for each duration. Since the 100-year return period storm is being developed, no adjustment to annual series need be considered. The adjusted values are plotted (Figure A.7), and the values for intervening intervals (1.5, 2.5 hours, etc.) are interpolated; these are shown in Column (4) of Table A.1. After division into increments and arrangement, the 100-year hypothetical storm is complete (last column of Table A.1) and can now be used for input to an HEC-1 model.



A.7. Depth-Duration Relationship for Study Area (Northeast Corner of Texas Panhandle)

Western U.S. This example describes the development of the two-year return interval storm for a 2.6 square-mile basin in Davis, California. Annual-series rainfall for a 3-hour storm duration and 10-minute rainfall interval is required.

The value of the 2-year return-period rainfall depth for the 6-hour duration is read from the appropriate figure in NOAA Atlas 2, Volume XI - California, as shown in Figure A.8. Since Davis lies in Region 4 (Sacramento-San Joaquin River Valley), the depth for the 1-hour duration is determined from the equation derived for that region given in NOAA Atlas 2.

$$Y_2 = 0.107 + 0.315\, X_1 \qquad \text{(A.1)}$$

where Y_2 is the 2-year 1-hour depth, and
X_1 is the 2-year 6-hour depth.

Substituting the value of 1.30 inches for X_1 in the equation gives a 2-year 1-hour depth Y_2 of 0.52 inches. Values for the depths of the 2- and 3-hour durations are determined from appropriate equations for Region 4 given in NOAA Atlas 2.

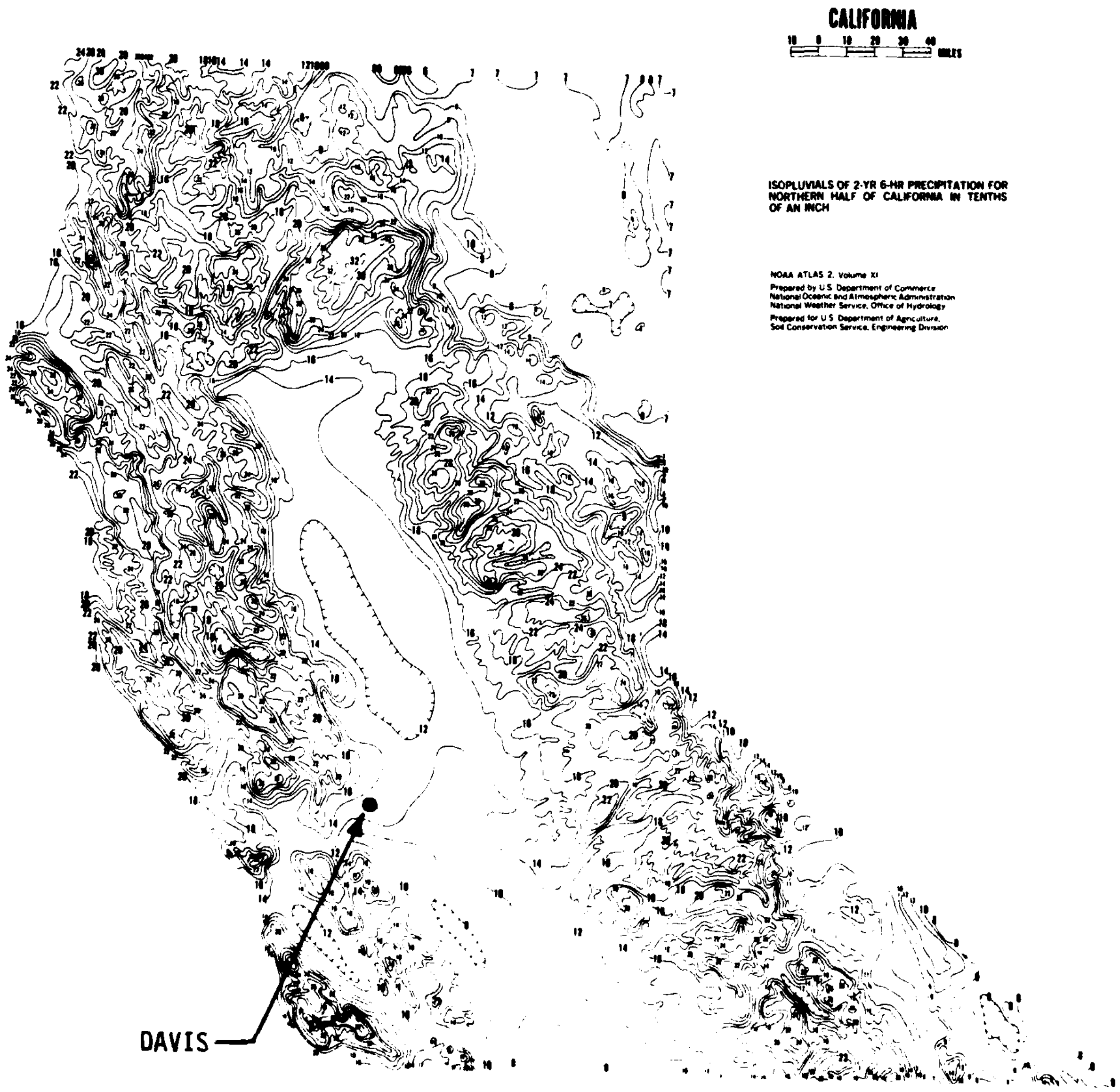
TABLE A.1

100-YEAR RAINFALL WITH TP-40, HYDRO-35

NE Corner of Texas Panhandle

Period (hours) (1)	Accum.1/ Point Rainfall (in.) (2)	Point Rainfall2/ Factor (100 mi^2) (3)	Accum. Depth (in.) (4)	Incre. Depth (in.) (5)	Arranged Incre. Depth (in.) (6)
0.5	2.62	0.615	1.61	1.61	0.06
1.0	3.46	0.723	2.50 4/	0.89	0.06
1.5			3.10 4/	0.60	0.06
2.0	4.35	0.810 3/	3.52	0.42	0.06
2.5			3.75	0.23	0.06
3.0	4.65	0.845	3.93	0.18	0.07
3.5			4.10	0.17	0.10
4.0			4.25	0.15	0.11
4.5			4.40	0.15	0.15
5.0			4.50	0.10	0.18
5.5			4.60	0.10	0.42
6.0	5.30	0.888	4.71	0.11	0.89
6.5			4.77	0.06	1.61
7.0			4.84	0.07	0.60
7.5			4.90	0.06	0.23
8.0			4.96	0.06	0.17
8.5			5.02	0.06	0.15
9.0			5.09	0.07	0.10
9.5			5.15	0.06	0.07
10.0			5.21	0.06	0.07
10.5			5.27	0.06	0.06
11.0			5.34	0.07	0.06
11.5			5.40	0.06	0.06
12.0	6.00	0.910 3/	5.46	0.06	0.06

1/ From TP-40, HYDRO-35.
2/ Figure A.3
3/ Interpolated from plot of 0.5-, 1-, 3-, 6-, 24-hour adjustments.
4/ Interpolated from Figure A.7 for intermediate values.



A.8. Rainfall Depth for 2-yr Return Interval,
6-hr Duration for Example (Davis, California)

$$2\text{-hr depth} = 0.240 \times (6\text{-hr depth}) + 0.760 \times (1\text{-hr depth})$$
$$= 0.240\ (1.30) + 0.760\ (0.52) = 0.71 \text{ inches}$$

$$3\text{-hr depth} = 0.468 \times (6\text{-hr depth}) + 0.532 \times (1\text{-hr depth})$$
$$= 0.468\ (1.30) + 0.532\ (0.52) = 0.89 \text{ inches}$$

Depth values for durations of less than 1 hour are found by applying the values in Table 12, Volume IX of NOAA Atlas 2, to the 1-hour depth. For this example, these values are:

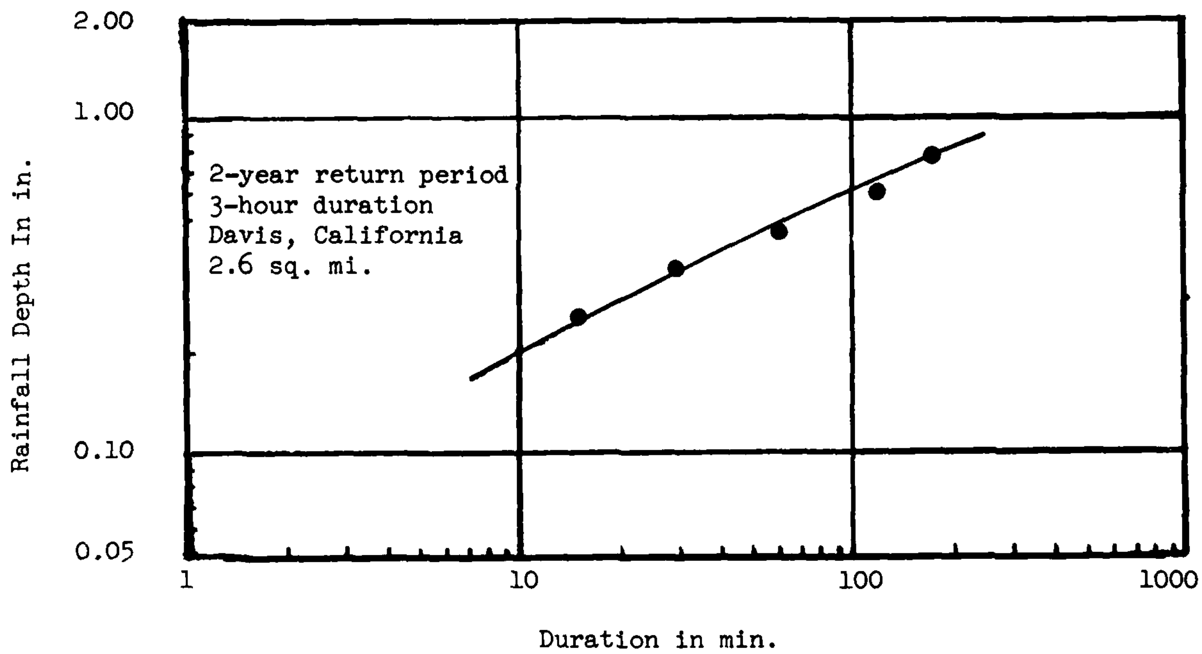
10-min-duration rainfall = 0.45 (0.52) = 0.23 inches
15-min-duration rainfall = 0.57 (0.52) = 0.30 inches
30-min-duration rainfall = 0.79 (0.52) = 0.41 inches

For a drainage area of 2.6 square miles the areal adjustment is judged to be negligible, therefore the point rainfall values are used. An adjustment from partial to annual series is desired, however, and the factor (0.88) for a two-year return period is taken from Table 2, Volume IX. The adjusted rainfall depths are given in the last column of Table A.2

TABLE A.2
SYNTHETIC 2-YEAR RAINFALL AT DAVIS, CALIFORNIA

Period	Accumulated Point Rainfall Depth (in.)	Areal Adjustment	Annual Series Adjustment	Adjusted Acumulated Depth (in.)
10 min	0.23	None	0.88	0.20
15 min	0.30	None	0.88	0.26
30 min	0.41	None	0.88	0.36
60 min	0.52	None	0.88	0.46
2 hr	0.71	None	0.88	0.62
3 hr	0.89	None	0.88	0.78

The final adjusted accumulated rain-depth values are plotted in Figure A.9. Values at 10-minute intervals are read from Figure A.9 and are incremented and arranged as shown in Table A.3. The last column represents the 2-year return period, 3-hour duration, 10-minute interval storm at Davis for input to an HEC-1 watershed model.



A.9. Rainfall Depth-Duration Relationship for Example (Davis, California)

TABLE A.3
3-HOUR-DURATION 10-MINUTE-INTERVAL 2-YEAR-RETURN-PERIOD STORM AT DAVIS, CALIFORNIA

Period (min)	Accumulated Rain Depth* (in.)	Incremental Depth (in.)	Arranged Storm (in.)
10	0.20	0.20	0.02
20	0.29	0.09	0.02
30	0.35	0.06	0.02
40	0.40	0.05	0.02
50	0.45	0.05	0.03
60	0.49	0.04	0.03
70	0.52	0.03	0.04
80	0.55	0.03	0.05
90	0.58	0.03	0.09
100	0.61	0.03	0.20
110	0.63	0.02	0.06
120	0.65	0.02	0.05
130	0.67	0.02	0.03
140	0.70	0.03	0.03
150	0.72	0.02	0.03
160	0.74	0.02	0.02
170	0.76	0.02	0.02
180	0.78	0.02	0.02

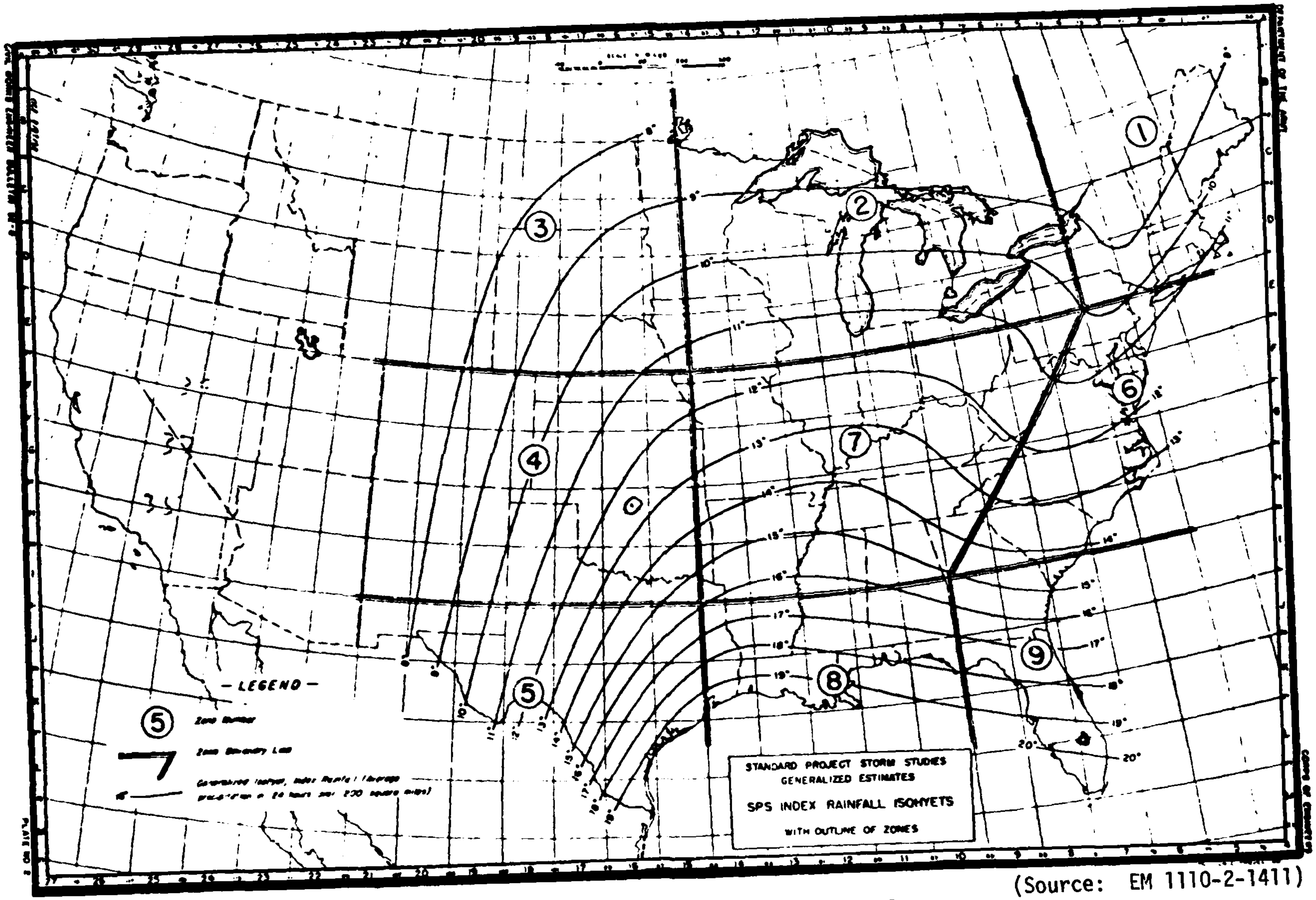
* From Figure A.9

A.5 Standard Project Storm.

The Standard Project Storm (SPS) is defined as that combination of severe meteorological events that gives the maximum precipitation reasonably characteristic of the geographic region of interest, excluding extremely rare events. Since the SPS is an infrequent event, no specific frequency can be assigned to it. It may range from a return interval of a few hundred years to a few thousand years. The SPS is often used as a design storm in which only a small degree of risk of exceedance can be tolerated such as in the design of an urban floodwall. It is usually used for comparison with the recommended protection for a particular project. Because the Standard Project Storm (SPS) is used mainly within the Corps of Engineers, only a limited number of publications describe its derivation and use, in contrast to materials available on hypothetical-frequency storms and the Probable Maximum Storm. Reference A.9 describes the SPS derivation for the United States east of 105^{o} longitude. SPS development for the remainder of the United States must be based on various published and unpublished Corps District reports and procedures. Only SPS derivation from reference A.9 is discussed further in this section.

SPS for Eastern and Central United States. Deriving the SPS for drainage basins greater than 1000 square miles requires special studies by the National Weather Service. The general criteria in EM 1110-2-1411 (reference A.9) are applicable to basins of less than 1000 square miles. The sequence of SPS derivation described in the reference includes: selection of an "index" rainfall, determination of the 24-, 48-, 72- and 96-hour SPS rainfall based on the index and the drainage area under study, adjustment of the rainfall for a basin shape factor, division into incremental rainfall amounts, and arrangement of the incremental rainfall values into the storm sequence.

(a) The SPS index rainfall is determined using Plate 2 of reference A.9 (reproduced here as Figure A.10). The rainfall depth determined from this figure represents the SPS rainfall for an area of 200 square miles and for a 24-hour duration. Further adjustment of this rainfall value is required, however, because a SPS is a 96-hour storm by definition, and because the drainage area under study will never be exactly 200 square miles.



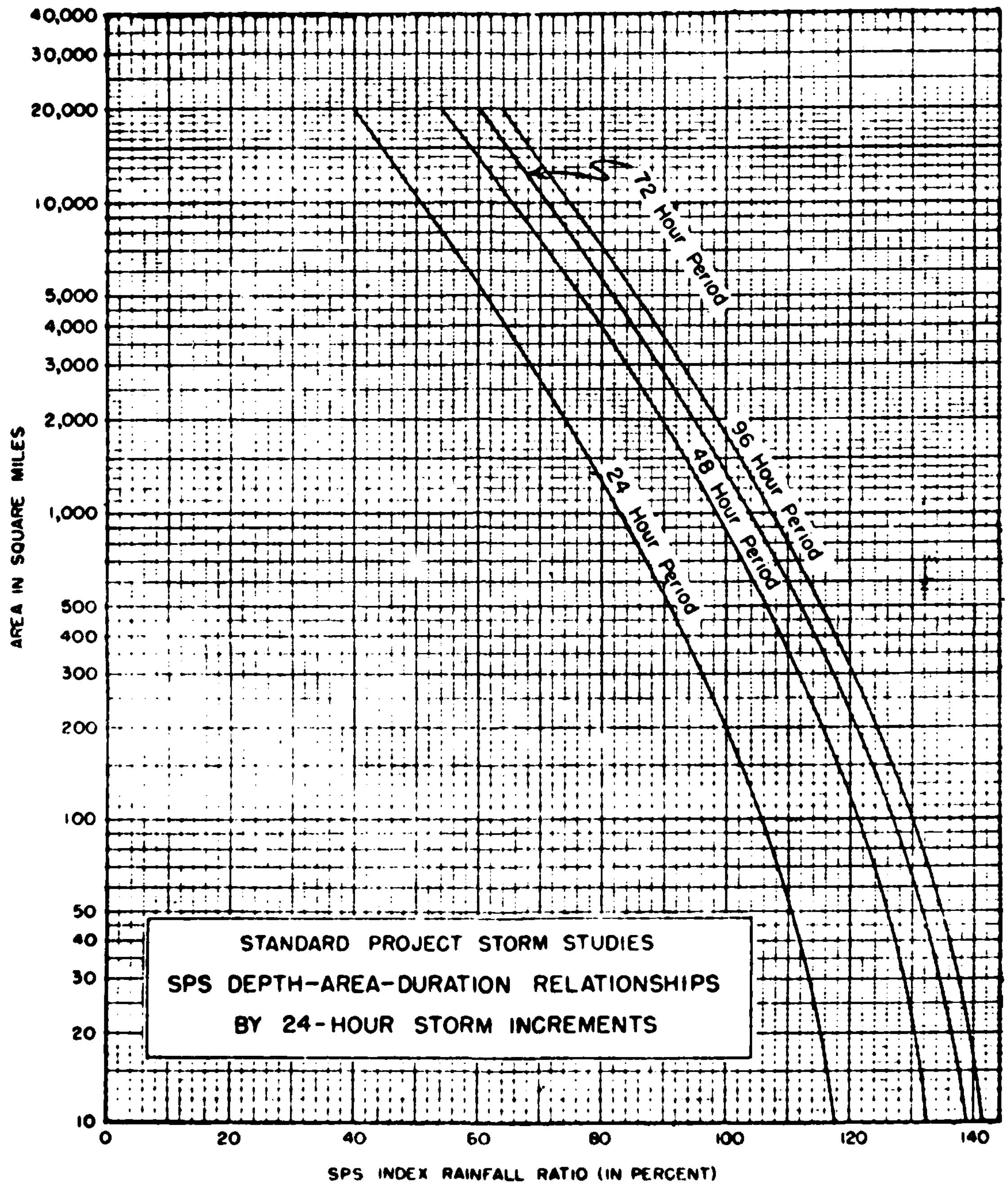
(Source: EM 1110-2-1411)

A.10. Index Rainfall for Standard Project Storm

(b) Plate 9 of reference A.9 (given here as Figure A.11) is used to convert the index rainfall to 24-, 48-, 72-, and 96-hour accumulated depths adjusted for the actual drainage area. The four values taken from the curves in the figure are each multiplied by the index to arrive at the accumulated rainfall values for each duration.

(c) These values are divided into total rain for each 24-hour period and ranked in the sequence 4, 2, 1, 3, with 1 being the highest 24-hour total. The percentage of the total 96-hour storm which each 24-hour period represents is then computed. The SPS criteria are developed using the symmetrical oval-shaped isohyetal pattern shown in Figure A.12 (Plate 12 of reference A.9). The rainfall values computed thus far are for a drainage basin that the symetrical pattern matches perfectly. The fit will always be imperfect in the real world, however, so an adjustment will be required for actual basin shape. That is done by overlaying the storm pattern on the basin, using a planimeter to compute total storm volume, and from this determining the maximum depth that could occur from the most severe arrangement of the pattern. The storm pattern may require several centerings to determine the maximum depth. Unless the drainage area is extremely small, the maximum 96-hour depth will be less than the 96-hour depth already determined. This maximum value of the 96-hour depth is then multiplied by the 24-hour percentages discussed at the start of this subparagraph. The result is the 24-hour SPS rainfall totals adjusted for the shape of the study basin.

(d) Division into Increments and Arrangement of Storm. Each 24-hour period is next subdivided into four 6-hour periods using the criteria in Figure A.13 (Plate 10 of reference A.9). Division into smaller increments uses a simple average of the rainfall in each 6-hour interval, except for the peak 6-hour period of the entire storm. Further breakdown of the peak 6-hour period to one-hour periods is made using the guidelines given in Figure A.14 (Plate 11 of reference A.9). It should be noted that the 1-hour percentages based on this time distribution have occasionally been found to give rainfall values that are not sufficiently conservative, particularly in the Southwestern Division of the Corps of Engineers (SWD). The percentages developed by the SWD for the 1-hour breakdown of the SPS have been added to Figure A.14.



A.11. Adjustment of Standard Project Storm Rainfall for Watershed Area

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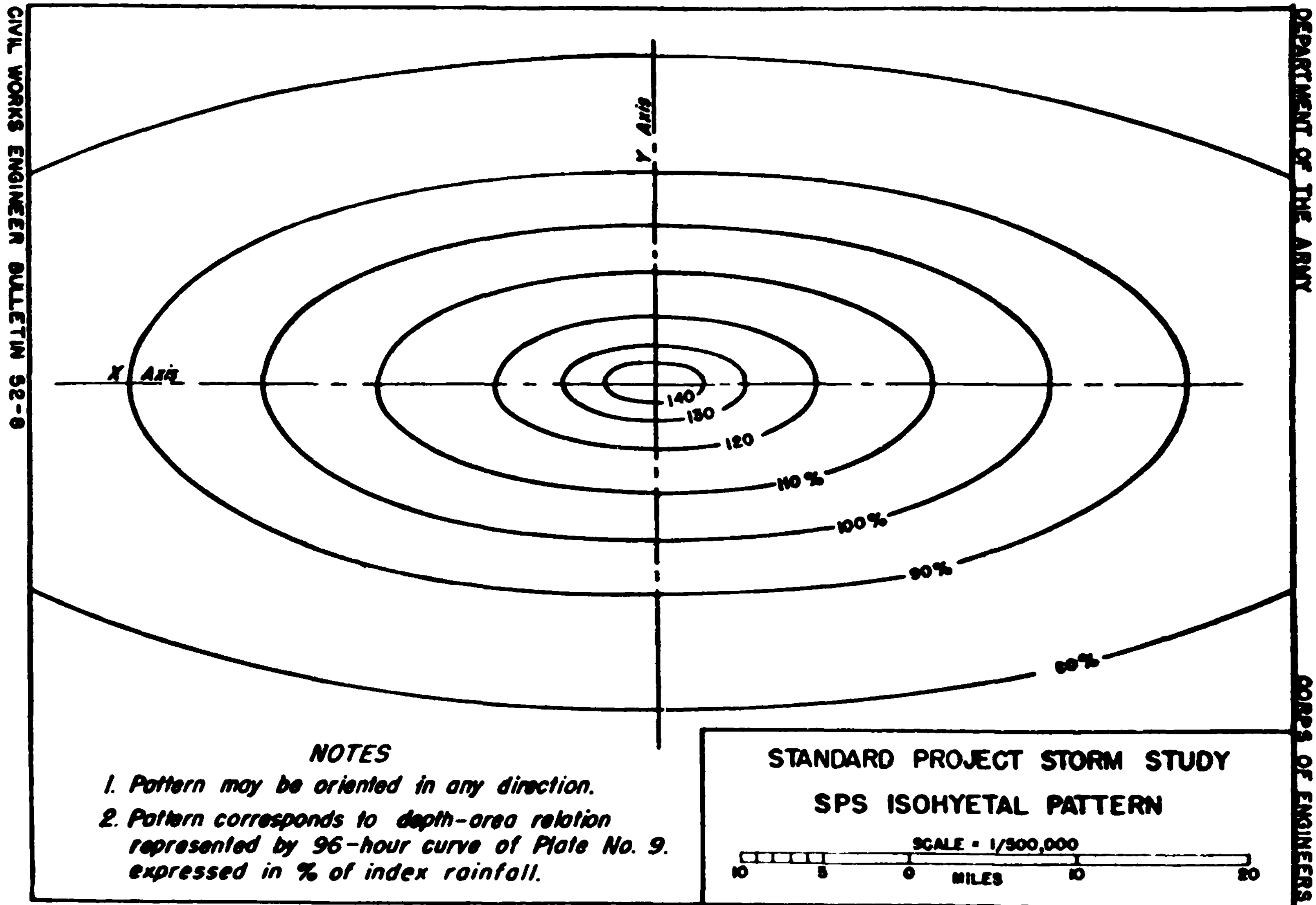

A.12. Isohyetal Pattern for Standard Project Storms

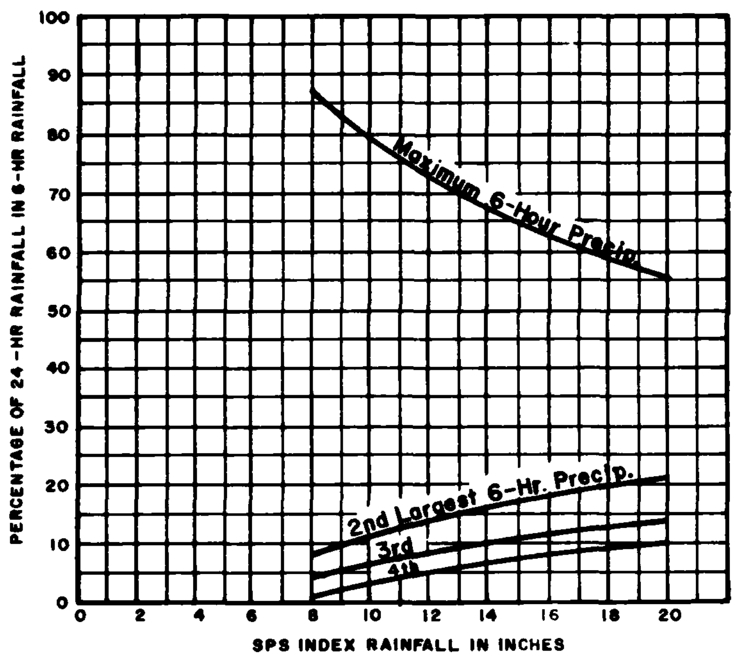

FIG.(a) SPS 24-HOUR PRECIPITATION OVER
200 SQ. MI. VS. PERCENT IN 6-HOUR PERIODS

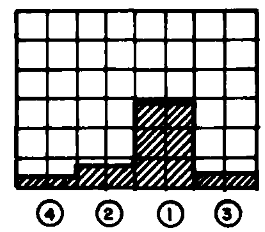

FIG.(b). TYPICAL ARRANGEMENT OF 6-HOUR RAINFALL QUANTITIES IN SPS

INDEX RAINFALL IN INCHES	PERCENTAGE OF 24-HOUR SPS RAINFALL IN DESIGNATED 6-HR PERIOD			
	④	②	①	③
1	2	3	4	5
8	1.0	8.0	87.0	4.0
9	2.1	9.5	83.0	5.4
10	3.2	11.0	79.2	6.6
11	4.3	12.3	75.9	7.5
12	5.3	13.8	72.5	8.4
13	6.1	14.9	69.6	9.4
14	7.0	16.0	66.9	10.1
15	7.6	17.0	64.5	10.9
16	8.1	17.9	62.4	11.6
17	8.8	18.9	60.3	12.0
18	9.1	19.7	58.5	12.7
19	9.8	20.3	56.8	13.1
20	10.1	21.0	56.1	13.8

FIG.(c). TABULATION OF DATA FROM FIG.(a)

A.13. Distribution of Standard Project Storm Rainfall in Time (6-hr Periods)

STANDARD PROJECT STORM STUDIES
GENERALIZED ESTIMATES

TIME DISTRIBUTION
OF 24-HOUR SPS RAINFALL

TIME DISTRIBUTION OF MAXIMUM 6-HOUR SPS RAINFALL

Rainfall Period (Sub-Division of 6-Hour Period)	Time Distribution of Maximum 6-Hour SPS Rainfall, Expressed in Percent of Total 6-Hour Rainfall — *Selected Unit Rainfall Duration, t_R				
	6-Hours	3-Hours	2-Hours	1-Hour	1-Hour (SWD)
#1	#2	#3	#4	#5	
1st	100	33	26	10	4
2nd		67	53	12	8
3rd			21	15	19
4th				38	50
5th				14	11
6th				11	8
TOTAL	100	100	100	100	100

*NOTE: The "selected unit rainfall duration," t_R is determined approximately from the synthetic unit hydrograph equation, $t_r = \frac{t_p}{5.5}$ in which "t_p" is the lag time from midpoint of unit rainfall duration, t_r, to peak of unit hydrograph, in hours, (See page 11, Engineering Manual for Civil Works, Part CXIV - Hydrologic and Hydraulic Analyses, Chapter 5 - Flood-Hydrograph Analyses and Computations). The following rounded-off values are to be used in the above table:

If t_p exceeds 16, use $t_R = 6$

If t_p is between 12 and 16, use $t_R = 3$

If t_p is between 6 and 12, use $t_R = 2$

If t_p is between 4 and 6, use $t_R = 1$

A.14. Hourly Distribution of SPS Rainfall in Maximum Six-Hour Period

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Intervals of less than one hour are sometimes needed for SPS ivision. Table A.4 gives a breakdown for various subdivisions of an hour d on a reapplication of the percentages taken from Figure A.13. Since no cial guidance is available for rainfall periods of less than one hour, edures throughout the country vary with local usage.

TABLE A.4
SUBDIVISION OF MAXIMUM 1-HOUR SPS RAINFALL*

	1-Hour	30-Min	20-Min	10-Min	5-Min
1st	100	33	26	10	5
2nd		67	53	12	5
3rd			12	15	6
4th				38	7
5th				14	7
6th				11	12
7th					26
8th					8
9th					7
10th					6
11th					6
12th					5

rived by extrapolation of peak interval percentages to increments of ess than 1 hour

(e) Example: Standard Project Storm Derivation.

SPS Derivation for Crab Orchard Creek, Illinois
Drainage Area = 289 Sq. Mi.
Computational time increment = 1 hr.

(1) SPS index rainfall taker from Figure A.10 is 13.3 in. (Plate 2 of reference A.9)

(2) Using the SPS ratios from Figure A.9 (Plate 9 of reference A.9) gives:

24 hr rainfall = 96% x 13.3 = 12.77 in.
48 hr rainfall = 112% x 13.3 = 14.90 in.
72 hr rainfall = 117% x 13.3 = 15.63 in.
96 hr rainfall = 121% x 13.3 = 16.09 in.

(3) Increment, arrange, and compute percent of total

Period	0-24 hr	24-48 hr	48-72 hr	72-96 hr
Accumulated Rain (in.)	12.77	14.90	15.63	16.09
Incremental Rain (in.)	12.77	2.13	0.73	0.46
Arranged Rain (in.)	0.46	2.13	12.77	0.73
Percent of Total	2.86	13.24	79.36	4.54

(4) Analyze alternative storm centerings for the storm pattern (Figure A-12), and determine the maximum depth over the basin

The maximum depth for this drainage area shape is 15.80 in.

(5) Apply 24-hour ratios to new depth

Period	0-24 hr	24-48 hr	48-72 hr	72-96 hr
Percent of Total	2.86	13.24	79.36	4.54
Adj. depths	0.45 in.	2.09 in.	12.54 in.	0.72 in.

(6) Using Figure A.13 (Plate 10 of reference A.9), break up the storm into 6-hour intervals.

For SPS index rainfall of 13.3 in., 24-hour rainfall distribution is:

	Percent of	Rainfall Distribution (in.)			
Period	96 hr Total	Day 1	Day 2	Day 3	Day 4
0-6 hr.	6.4	0.03	0.13	0.80	0.05
6-12 hr.	15.3	0.07	0.32	1.92	0.11
12-18 hr.	68.7	0.31	1.44	8.62	0.49
18-24 hr.	9.6	0.04	0.20	1.20	0.07

(7) Break up the maximum 6-hour period into hourly periods to complete SPS derivation, using the guidelines in Figure A.14 (Plate 11 of reference A.9). One-hour percentages of the peak 6-hour rainfall are: 10%, 12%, 15%, 38%, 14%, and 11%; the resulting distribution of hourly rainfall is shown in Table A.5.

TABLE A.5

FINAL 1-HOUR SPS RAINFALL DISTRIBUTION

Period (hr)	Hourly Rainfall (in.)
0-6	.03/6
6-12	.07/6
12-24	.31/6
18-24	.04/6
24-30	.13/6
30-36	.32/6
36-42	1.44/6
42-48	.20/6
48-54	.80/6
54-60	1.92/6
61	.86
62	1.03
63	1.29
64	3.28
65	1.21
66	.95
66-72	1.20/6
72-78	.05/6
78-84	.11/6
84-90	.49/6
90-96	.07/6

A.6 Probable Maximum Storm

The Probable Maximum Storm (PMS) is defined as the most severe combination of meteorological occurrences considered reasonably possible in a particular region. It is felt to be an upper limit of flood-producing rainfall (or snowpack melting when applicable) and is used as a design storm where virtually no risk of flooding can be tolerated. The PMS has historically been used in dam design to ensure the adequacy of spillways and top-of-dam elevations for high dams.

As with the hypothetical frequency storms, one set of generalized criteria is applied to the majority of the U.S., and a variety of regional criteria, accounting primarily for orographic effects, to the balance. Details for constructing a PMS for a particular region are given in the various Hydrometeorological Reports and Technical Papers listed in the references. The reports, HR-51 and HR-52, for the United States east of the 105th meridian, apply to most of the country and are discussed further here.

The mechanics of storm breakdown and arrangement presented in these reports are similar to methods for the western United States presented in other publications.

PMS for the Eastern and Central United States. The steps for deriving a PMS, using Probable Maximum Precipitation data from HR-51 (reference A.16) and data for the determination of shape, orientation, and distribution from HR-52 (referrence A.16a), are as follows:

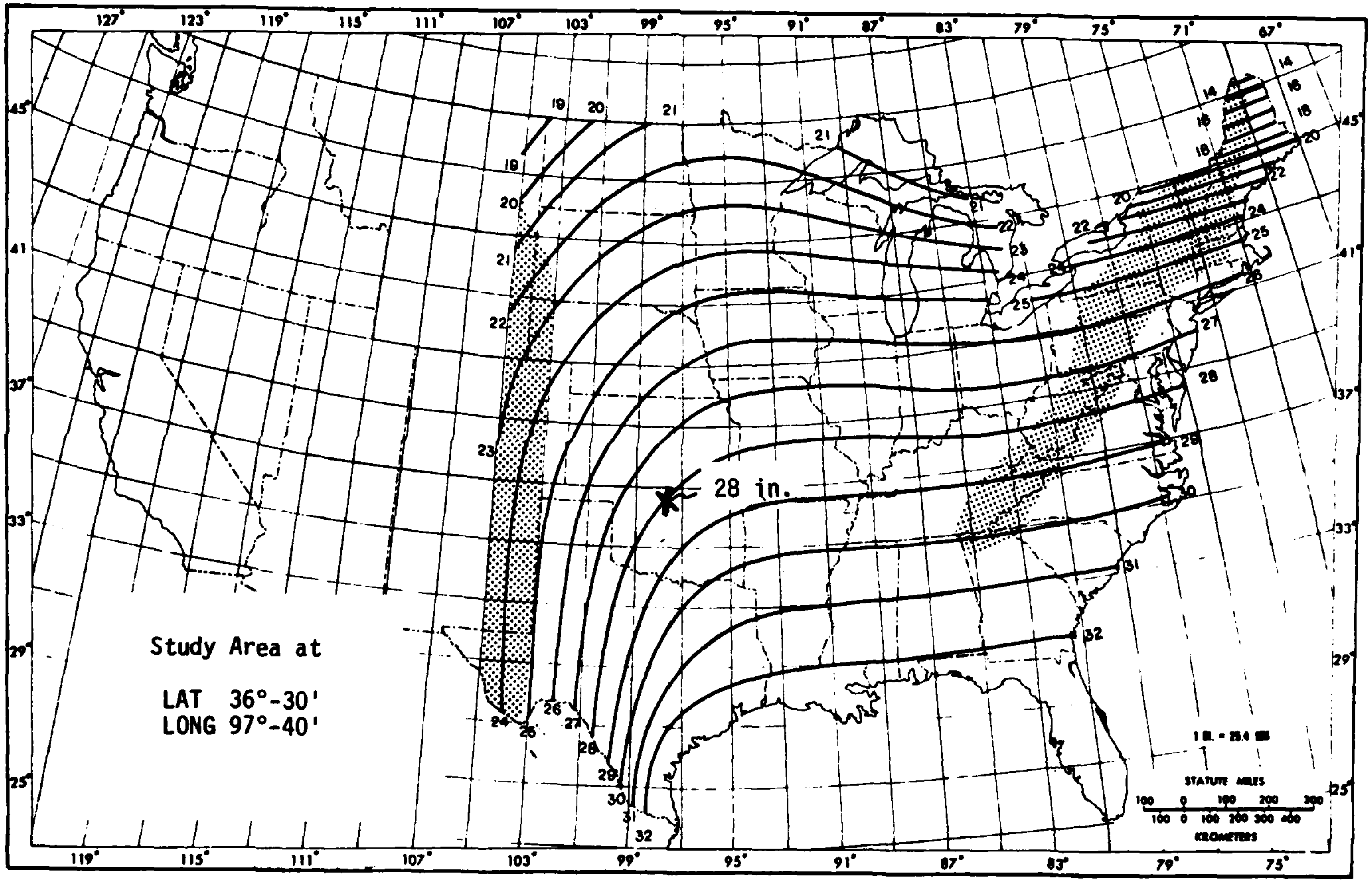
(a) Determine isohyetal Probable Maximum Precipitation (PMP) values for the study area for desired drainage area sizes (10, 100, 200, 1,000, 5,000, 10,000, 20,000 square miles) and for corresponding storm durations (6-, 12-, 24-, 48-, and 72-hours) using the appropriate plates from the report. For example, Figure A.15 shows how the 6-hour value is determined.

(b) Plot a family of duration curves (6-, 12-, 24-, 48-, and 72-hours) for PMP intensities versus drainage area size on semi-logarithmic paper, as shown in Figure A.16.

(c) Interpolate for the desired storm area, determine PMP intensities for each duration and plot on ordinary graph paper (Figure A.17).

(d) Determine rainfall in each 6-hour time interval by interpolation, then increment the rainfall by successive subtractions. The PMP will be the maximum value for the selected storm area only; areas greater or less than the selected storm area will show smaller values of PMP. Considerable trial and error computations will normally be required to determine the storm area which maximizes average precipitation in the study watershed.

(e) The "Hop Brook adjustment" has often been applied to PMP values in Corps studies for the Eastern and Central United States. It attempts to adjust for the reduced probability of a Probable Maximum Storm occurring directly over a small watershed. The smaller the watershed, the less is the likelihood of such a large storm occurring directly over it. Factors for reduction of PMP intensities for various drainage areas are shown on Table A.6, taken from reference (A.19).



(Source: National Weather Service)

All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

A.15. Example of Probable Maximum Precipitation Intensities

TABLE A.6
HOP BROOK ADJUSTMENT FACTORS*

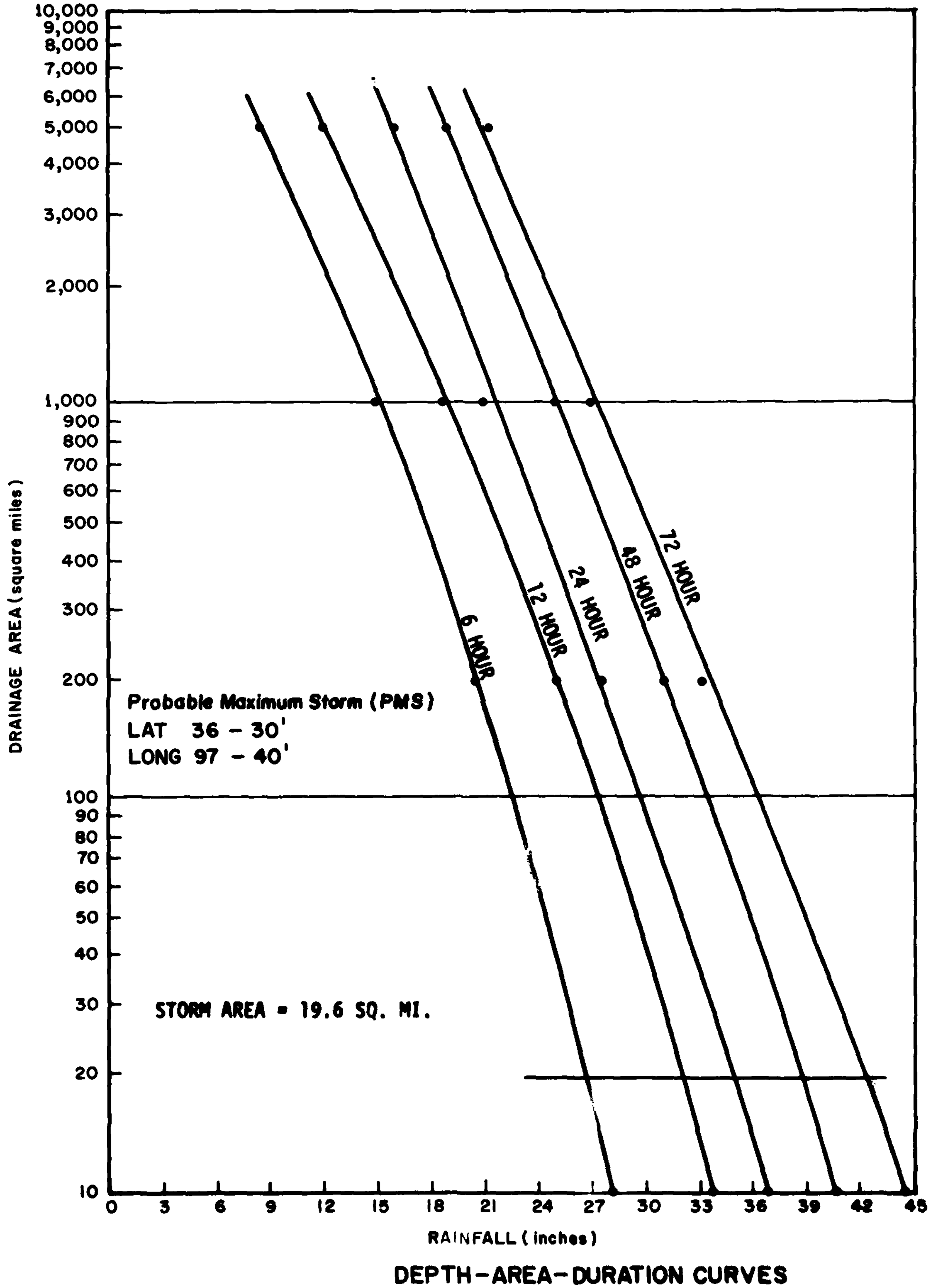
Drainage Area (sq mi)	Adjustment Factor
1000	.9
500	.9
200	.89
100	.87
50	.85
10	.80
1	.80

*Note: Not all Corps of Engineer districts apply this adjustment factor.

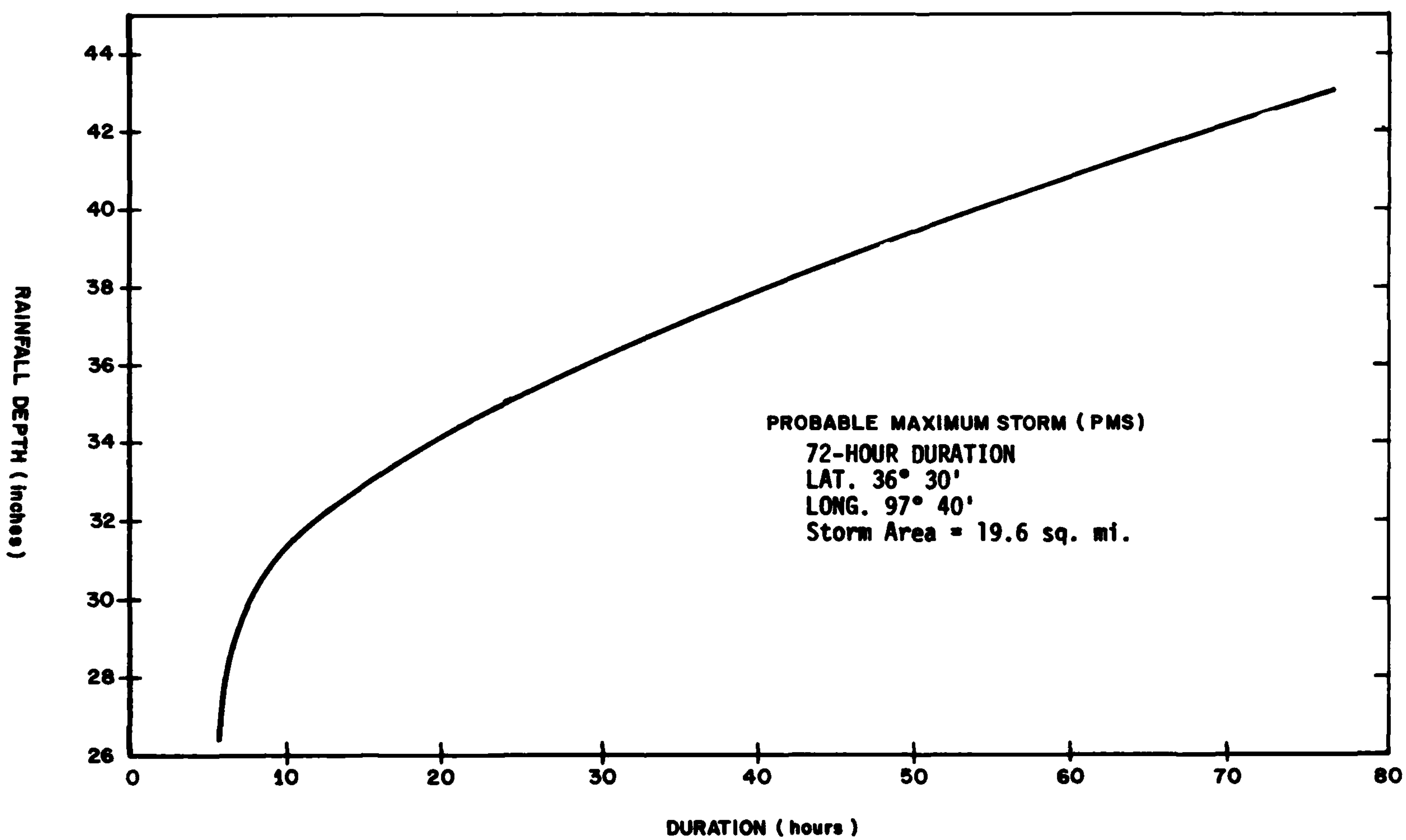
(f) Studies for HR-52 have found that major storms have a dominant orientation, which may or may not be similar to the general orientation of the watershed. The PMP will often be reduced, depending on the drainage area size and the angle between the storm and watershed orientation. No reduction is taken for orientation differences less than ±40.°, regardless of area, or for drainage areas less than 300 square miles, regardless of orientation. Maximum PMP reduction due to orientation is 15 percent.

(g) Storm shape is given by criteria in HR-52, with the PMS having a general elliptical isohyetal pattern with a ratio between major and minor axes ranging from two to five. Areas less than 300 square miles may use a circular shape, if desired. The adopted ratio of the axes will be that which gives the most hydrologically-severe storm (one which maximizes volume) within the study watershed. This usually requires significant trial and error work.

(h) With the maximized storm pattern established, the spatial variability of the precipitation is determined, again by HR-52, through figures and nomographs. Spatial variability is greatest for the maximum six-hour period, diminishes for the second and third greatest six-hour periods, and has no variability for the remaining six-hour periods. Precipitation profiles are given in HR-52 to develop the spatial variation in the peak 18 hours of the PMS.



A.16. Depth-Area-Duration Curves for Example Probable Maximum Storm



DEPTH–DURATION RELATIONSHIP

A.17. Rainfall Depth vs Duration for Probable Maximum Storm

(i) Final development of the PMS calls for a temporal arrangement to give the most critical hydrologic response (maximum runoff). The six-hour values of the PMS are arranged such that they decrease progressively to either side of the greatest six-hour value. The four greatest six-hour increments arre placed at any position in the sequence except during the first 24 hours. No guidance is yet available to develop PMS increments of less than six-hours. The application of SPS criteria may be appropriate for the greatest six-hour PMS increment, with each of the other six-hour PMS increments being averaged.

Only the general outline of PMS development has been given in this appendix. The engineer should refer to HR-51 and HR-52 for detailed guidance in determination of the PMS.

A.7 Supplemental Information

Additional material to consider in deriving these storm data which is also applicable to hypothetical storm calculation include:

Extrapolation of Frequency Data. The Technical Papers of the NWS indicate that a limited extrapolation (to 200-year return periods) is appropriate based on the available generalized data. However, extrapolation to a 500-year return period, as is often required for the hydrology necessary in flood-insurance studies, is of questionable validity. Rainfall for the extrapolated 500-year event may be only 20 to 30% greater than the 100-year total, but significantly less than the SPS for the same storm duration. While an SPS cannot be assigned a specific frequency of exceedance, it is likely that it is of the same order of magnitude as a 500-year event. An evaluation of the 500-year rainfall should include an examination of the SPS rainfall to assist in development of appropriate estimates of the 500-year event. An adjustment of an extrapolated 500-year rainfall total may be necessary to ensure reasonable compatibility with the SPS.

Urbanization effects. Meteorological studies have shown an increase in the number and intensity of thunderstorm rainfall events for watersheds downwind from major urban areas (population greater than one million). Studies of downwind rainfall for Chicago, St. Louis, Detroit, Washington,

Houston, New Orleans, and Cleveland have shown an increase in warm-season rainfall ranging from 10% at New Orleans, to 25% at Chicago for areas up to 30 miles downwind from the city. A study of St. Louis weather patterns (reference A.19) found that the number of heavy rainstorms has increased dramatically since 1960, with 5-minute rainfall rates increased by at least 50% over large downwind areas. While there is currently no direct way of incorporating urbanization effects into rainfall estimates, the existance of this increase should be recognized where the study watershed falls within the sphere of urban influence. Measured rainfall data could be used to supplement the generalized data which, along with conservative selection of loss rates, could account for some of the urbanization effects on rainfall. Sensitivity tests using increased rainfall amounts could be performed to evaluate this approach.

Hypothetical storm calculation within HEC-1. Once the individual is familiar with the derivation of hypothetical storms, the potential exists to perform many of the calculations with HEC-1. The current version of the computer program can develop the SPS and PMS for areas east of the 105th meridian, with the user-supplied point rainfall index and shape factor. The latest version of HEC-1 incorporates procedures for the development of annual-series hypothetical-frequency storms for the user-supplied point rainfall depth-duration array. Adjustments for depth, area, and annual series are made automatically, incremental depths are determined, and the entire storm pattern is arranged within the program.

A.8 References

(A.1) Technical Memorandum NWS HYDRO-35, "Five to 60 Minute Precipitation Frequency for the Eastern and Central U.S.," 1977.

(A.2) NWS TP 40, "Rainfall Frequency Atlas of the United States, 30-minute to 24-hour Durations, 1 to 100 Year Return Periods," 1961.

(A.3) NWS TP 49, "Two to Ten Day Precipitation for Return Periods of 2 to 100 Years in the Contiguous United States," 1964.

(A.4) NOAA Atlas 2, "Precipitation Frequency Atlas of the Western United States," Volumes 1-13, 1973.

(A.5) NWS TP 43, "Rainfall Frequency Atlas for the Hawaiian Islands, Durations to 24 Hours, Return Periods to 100 Years," 1962.

(A.6) NSW TP 51, "Two to Ten Day Rainfall for Return Periods of 2 to 100 Years in the Hawaiian Islands," 1965.

(A.7) NWS TP 47, "Probable Maximum Precipitation and Rainfall Frequency Data for Alaska, Durations to 24 Hours, Return Periods to 100 years, 1963.

(A.8) NWS TP 52, "Two To Ten Day Precipitation for Return Periods of 2 to 100 Years in Alaska," 1965.

(A.9) EM 1110-2-1411 (Formerly EB 52-8), "Standard Project Flood Determination," revised March 1965. SPS procedures for the eastern and central U.S. east of 105^{0} longitude and for basins less than 1000 square miles.

(A.10) HR 33 "Seasonal Variation of Probable Maximum Precipitation (PMP), East of the 105th Meridian for Areas 10 to 1000 Square Miles and Durations of 6 to 48 hours," 1956. Applies to areas east of the Rockies.

(A.11) HR 36, "Interim Report on Probable Maximum Precipitation in California," (1961). For drainage areas less than 5000 square miles.

(A.12) TP 38, "Generalized Estimates of PMP for the U.S. West of the 105th Meridian for Areas Less Than 400 Square Miles and Durations to 24 Hours," 1960.

(A.13) HR 39, "Probable Maximum Precipitation in the Hawiian Islands," May 1963.

(A.14) HR 43, "Probable Maximum Precipitation, Northwest States," June 1965. Estimates of PMP for durations of 1 to 72 hours for basins from 10 to 5,000 square miles west of the Cascade Divide and 10 to 1000 square miles east of the divide in the Columbia River Basin.

(A.15) HR 49, "PMP Estimates, Colorado and Great Basin." Covers areas of California, Nevada, Utah, Arizona, Wyoming, Colorado and New Mexico for durations of 15 minutes to 6 hours and areas from 1 to 500 square miles.

(A.16) HR 51, "Probable Maximum Precipitation Estimates, United States East of 105th Meridian," 1974.

(A.16a) HR 52, "Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian," 1981.

(A.17) EC 1110-2-163 (Draft Engineering Manual), "Spillway and Freeboard Requirements for Dams, Appendix C, Hydrometeorological Criteria and Hyetograph Estimates," U.S. Army Corps of Engineers, August 1975.

(A.18) EM 1110-2-1406, "Runoff from Snowmelt," U.S. Army Corps of Engineers, 1960.

(A.19) Changnon, S. A., "Rainfall Changes in Summer Caused by St. Louis," Science, Vol. 205, 27 July 1979, pp. 402-404.

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